02 Apr 1995, 1:00 pm - 6:00 pm

Discussions and Replies — State-of-the-Art

Multiple Authors

DISCUSSIONS AND REPLIES STATE-OF-THE-ART

Discussion on paper titled: "Deformation Characteristics of Soils and Soft Rocks under Monotonic and Cyclic Loads and Their Relationships," by F. Tatsuoka, D. Lo Presti, and Y. Kohata, Paper No. SOA1.

By Vincent P. Drnevich and Alaa K. Ashmawy, Purdue University, W. Lafayette, IN, USA.

The authors are to be commended in producing an excellent report. The writers believe that much valuable data and many keen insights are presented. The section on **Damping Ratio** is of particular interest to the writers. The authors correctly list a number of parameters that affect damping ratio but fail to include loading stress path. It can be shown that, like modulus, damping ratio in axial loading is different from damping ratio in torsional loading. For a given specimen, damping ratios obtained from axial loading are less than those for torsional loading. Hardin (1965) suggests that for low strain tests on sand, the values from axial loading are approximately 3/4 those from torsional loading. The writers suggest that all future reporting of damping ratio in the literature include the type of test used in making the determination and appropriate subscripts be used to identify the type of test, e.g., D_L for axial loading and D_T for torsional loading which is used in ASTM D4015-92, Standard Test Methods for Modulus and Damping of Soils by the Resonant-Column Method.

The authors are commended for presenting the data on strain rate effects in Figures 46 and 49. The writers would like to emphasize the fact that strain rate is a much more appropriate parameter than frequency of loading for two reasons. 1) Like strain, strain rate is often considered a variable in constitutive modeling of materials; 2) The term "frequency " is associated with periodic motion and may involve different wave forms. Therefore, frequency could be a deceiving parameter. For instance, strain rates associated with square and sinusoidal waves at the same frequency and amplitude are vastly different. Even for similar wave forms, vastly different frequencies could have similar strain rates. Consider a cyclic torsion test with sinusoidal loading at 1 Hz and single amplitude strain of 0.1%. The magnitude of strain rate associated with the test is approximately 2.4%/min, as shown in Figure 1a below. Consider next a torsional resonant column with a resonant frequency of 100 Hz and a single amplitude strain of 0.0001%. The corresponding strain rate magnitude, as shown in Figure 1b, also is 2.4%/min. This example shows the importance of considering strain rate rather than frequency. It also shows that results from "dynamic" tests provide data at strain rates typically associated with "quasi-static" loading.

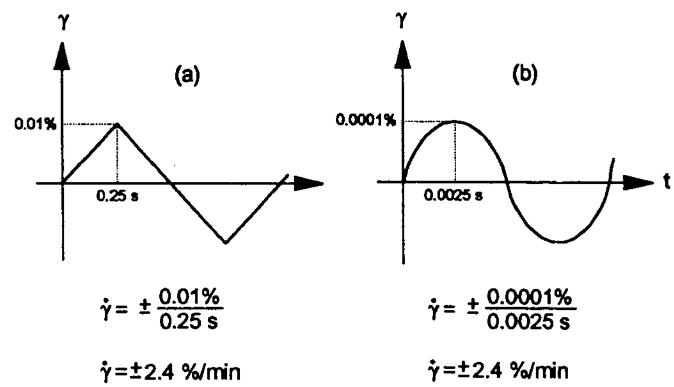

Figure 1. Effect of frequency and amplitude on strain rate in "quasi-static" and "dynamic" testing.

It may be possible to get a better feel for strain rate effects by use of a three-dimensional plot where damping ratios (in the z direction) are plotted vs strain and strain rate (in the x and y directions, respectively). This can be particularly useful when wide ranges of strains and strain rates are presented. Figure 2 below shows a three-dimensional plot with data for undrained tests obtained from Figure 49 in the authors' paper.

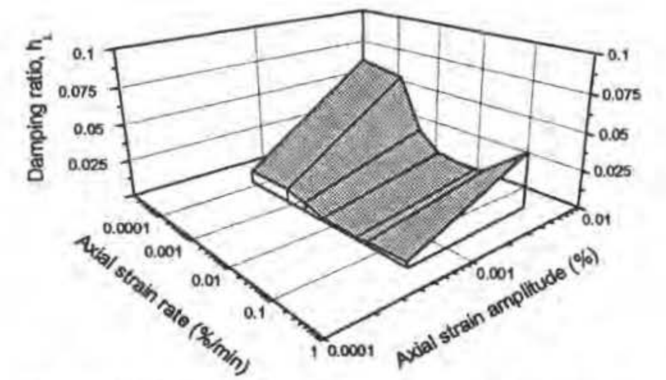

Figure 2. Effect of strain rate and amplitude on damping ratio (3-D representation)

Reference:

Hardin, B.O., 1965, "The Nature of Damping in Sands," JSMFD, ASCE, Vol.98, No. SM7, pp.667-692.

Discussion on paper titled "Deformation Characteristics of Soils and Soft Rocks under Monotonic and Cyclic Loads and their Relationships", by F. Tatsuoka, D. Lo Presti & Y. Kohata, Paper SOA Session 1

By: Pierre-Yves Hicher, Laboratory of Mechanics Soil, Structures, Materials, CNRS URA 850, Ecole Centrale de Paris, France.

Tatsuoka, Lo Presti and Kohata present a very comprehensive study on the mechanical properties of geomaterials (cohesive and non cohesive) at small strains. The first part is devoted to comparing strain measurements made either outside or inside the triaxial cell. In the latter case measurement devices were placed directly on the specimen, in their central part, or fixed at each end. The results, obtained by different laboratories, show clearly the effect of the experimental process. The conclusion is that if one wants to measure precisely strains lower than 1%, the only reliable system is the disposal of strain measurement devices in the central part of the specimen inside the triaxial cell. My own studies on the subject (Hicher 1985, Biarez and Hicher 1994) lead me to the same conclusions. This is even more accurate when the specimen are stiffer because of the increasing influence of the system compliance and of the effects of the contacts at the top and base. Also essential are the local strain measurements in the case of lubricated ends. The influence of end restrain on triaxial test results has now been widely demonstrated and the use of lubricated ends to avoid samples disturbances and in particular early strain localisation has been developed. Due to significant deformability of these systems during triaxial loading, accurate measurement of stress-strain relationship can be achieved only with local gage. As an example, we present in Figure 1 the results obtained in a soft clay by using outside and inside strain gages. The results were significantly different and the differences increased with the stiffness of the specimen.

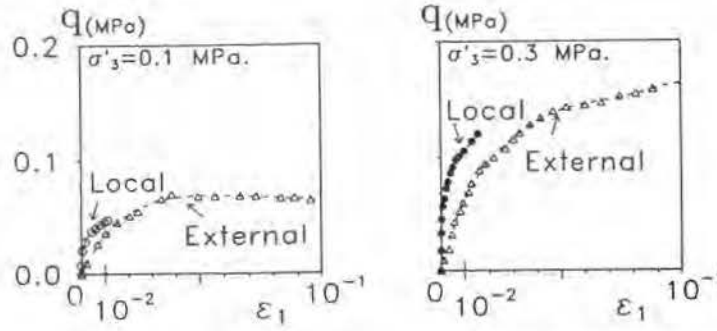

Figure 1. Comparison between external and local measurements of vertical strain in a drained triaxial test on a normally consolidated clay.

The data obtained with local strain gages indicate that an elastic behaviour can be reached for strain amplitudes lower than 10^{-5} in almost all geomaterials. Ths complies with the results of other studies which conducted either the same kind of tests (Hicher et al 1987) or which used other apparatuses, mainly resonant columns (Hardin and Drnevich 1972). The analysis of strain rate influence on the Young moduli leads the authors to conclude that the behaviour is essentially elastic, which little effect of the strain rate. As a consequence the elastic stiffness measured by means of static or dynamic tests is essentially the same. This idea has already been pointed out and demonstrated by others, but it is a very important one, which lead to a more unified way of studying soil behaviour and design procedures. I share these conclusions, and especially on essentially non-viscous materials like granular soils (sands and gravels). For finer soils, like clays and even silts, my own analysis (Hicher 1985) and those of others (Stokoe and Isenhower 1981) have demonstrated the influence of viscous properties also on the initial stiffness. Figure 2 presents some results obtained during cyclic undrained triaxial tests at different frequencies on a soft clay using local gages. One can see that even at strains lower than 10^{-5} where the equivalent modulus can be considered as constant, there is an influence of the strain rate and this influence appears to be of the same amount of magnitude as it is for larger plastic strains. These same results and others of the same kind (Hicher 1992) or those obtained by using a longitudinal resonant column (Rivera 1988) indicate also that the initial damping ratio is very small for sands (around 1%) compared to the values obtained on clays (several percents according to the clay structure), showing a viscous behaviour at very small strains.

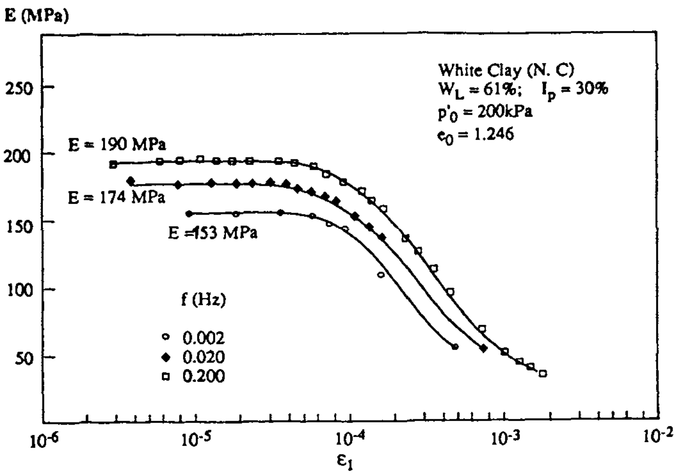

Figure 2. Influence of strain rate in cyclic triaxial tests on a soft clay.

The measurement of the damping ratio with the strain rate on clays presented by the authors (Figures 46 to 49) are however not very convincing to me, especially because the interpretation of "partially drained" tests are difficult to make, the test conditions been no more homogeneous. The explanation of the creep effect (which should be included in the general viscous behaviour) in Figure 46 leads to the comparison of damping ratios at different strain amplitudes instead of comparing them at the same strain amplitude as it is done in the D-ϵ curves.

However the strain rate influence is not big enough to modify the afore-mentionned similarities between "static" and "dynamic" moduli, and I particularly agree that the comparison which is obtained between static triaxial tests on intact samples at very small strains and field-seismic surveys is very reliable. El Hosri et al (1981) present an example of comparing these two types of results obtained in a stiff marl, foundation soil of a nuclear power plant . The results here agree very well, even if the stiffness measured in triaxial tests is systematically lower than the one measured by cross-hole techniques in the field, due probably to the strain rate effect but also due to disturbances during boring. The latter point is very important when one wants to measure the behaviour of soil samples at small and very small strains, because these disturbances affect mainly the initial part of the stress-strain curves and are progressively overcome by strain hardening during test loading.

Elastic properties of isotropic materials are independant of test conditions (triaxial test and torsional shear test for example). But this is no longer the case where plastic strains are concerned. The authors present some comparisons between E/Emax - ϵ (or G/Gmax - γ) curves obtained from triaxial and torsion shear tests which illustrate this. We can also see an example in Figure 3 which shows a more accentuated decay of the curves during a monotonic loading than during a cyclic loading on a dry sand (Biarez and Hicher 1994). The number of cycles at each stage and the conditions of drainage are also influential parameters on the strain dependancy of stiffness. In particular the cumulative pore pressure at each loading stage can strongly influence the results for strain amplitude higher than 10^{-3}. Therefore one has to be careful in interpreting these curves for the purpose of simplified methods using for example non linear elasticity (or viscoelasticity).

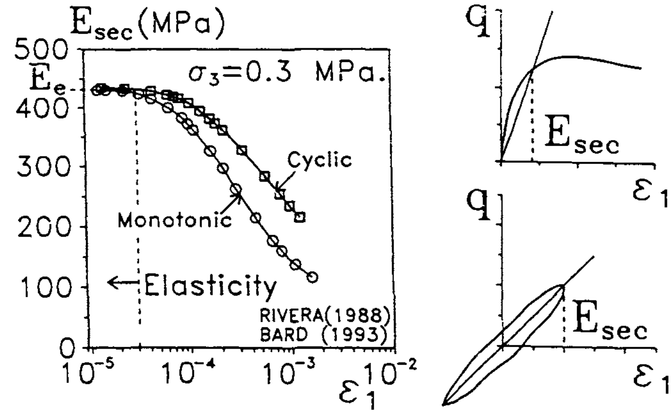

Figure 3. Decrease of secant modulus with strain amplitude during monotonic and cyclic triaxial tests on a dry Hostun sand

The study of a preshearing effect (for strain amplitudes lower than 10^{-3}) leads to interesting results : one can see in particular that, while there is little effect on the initial stiffness, the damping ratio is very much affected and its values are significantly reduced. At the same time the liquefaction strength increases considerably. It seems here that the evolution of the damping ratio is a more valuable indicator of the structure evolution than is the elastic modulus. The damping ratio represents mainly the plastic dissipation of the material during cyclic loading. In this sense an increase of its value indicates a reduction in the stability of the soil structure. Larger plastic strains which can develop during cyclic loading might lead to liquefaction in the case of undrained tests.

In conclusion, I would like to say that this paper is a valuable contribution to a global understanding of the mechanical behaviour of geomaterials. Both static and dynamic tests are valuable tools in approching this behaviour. One should not hesitate to use dynamic tests for studying static cases, as in evaluating soil displacements around civil engineering structures. The definition of the range of strain within which the modulus has to be determined bears a great influence on the results. This strain range is usually smaller than 10^{-2} and often smaller than 10^{-3} and some dynamic tests are well adapted for the accurate measurements of these small strains. Static tests can equally well be adapted for the determination of the soil characteristics under dynamic conditions, as machine vibrations or seismic loadings. The triaxial test with local strain measurement, as discussed above, presents the big advantage, contrary to dynamic tests, of being able to give access to the direct measure of the stress and strain tensors in a large range of strain amplitude.

References

Biarez J & Hicher P.Y (1994) Elementary Mechanics of Soil Behaviour, ed. Balkema 200 pp
El Hosri M.S., Biarez J. & Hicher P.Y (1981) Dynamic triaxial and vibratory in-situ behavior of cohesive soil, Int. Conf. on Recent Advances in Geotechnical Earthquake Eng. and Soil Dynamics, Univ. Missouri-Rolla.
Hardin B.O. & Drnevich V.P. (1972) Shear Modulus and Damping in Soils; Measurement and parameters effects, Journal of the Geotechnical Engineering Division, ASCE, vol. 98, SM 6, pp. 603-624.
Hicher P.Y. (1985) Comportement mécanique des argiles saturées sur divers chemins de sollicitations monotones et cycliques. Application à une modélisation élastoplastique et viscoplastique, Thèse d'Etat, Univ. Paris 6.
Hicher P.Y., El Hosri M.S. & Homsi M. (1987) Cyclic Properties of Soils within a large Range of Strain Amplitude, 3rd Int. Conf. on Soil Dynamics and Earthquake Eng., Princeton.
Hicher P.Y. (1992) élasticité dans les géomatériaux non cohérents, comptes-rendus du GRECO Géomatériaux.
Rivera R.C. (1988) Détermination des propriétés mécaniques des sables et des argiles en régime dynamique et cyclique en faibles déformations, thèse de l'Ecole Centrale de Paris.
Stokoe K.H. & Isenhower (1981) Strain rate dependant sherar Modulus of San Francisco Bay Mud, Proc. Int. Conf. on Recent Advances in Geo. Earthquake Eng. and Soil Dyn., Univ. Missouri-Rolla pp 597-602.

Discussion on the state-of-the-art paper titled "Deformation Characteristics of Soils and Soft Rocks Under Monotonic and Cyclic Loads and Their Relations", by F. Tatsuoka, D. Lo Presti and Y. Kohata.

By: Vicente Cuéllar, Laboratorio de Geotecnia, Centro de Estudios y Experimentación de Obras Públicas, Spain.

First, I would like to congratulate the authors on their excellent state-of-the-art report and the vast amount of experimental results on which they have based their conclusions.

To explain the difference in the values of damping ratio measured locally with LDT and globally with external gages in cyclic loading triaxial tests, the authors refer to Fig. 5 c) and d) where h-log$(\epsilon_a)_{SA}$ relations obtained with both devices from samples of dense and loose Toyoura sand are plotted respectively. In my opinion horizontal parts on those plots such as the one exhibited by the LDT plot in Fig. 5 c) are difficult to justify theoretically. Similarly, sections on the proximeter curves below LDT curves such as that for strains larger than $2\text{-}10^{-4}$ in Fig. 5 d) are meaningless. However, the assumptions made by the authors to estimate damping values in the small strain range using Eq. 7 seem reasonable, and their efforts to obtain further insight into an area where much experimental work of that kind is needed is highly appreciated.

They also suggest, on the basis of the results obtained after the cyclic prestraining of different sand samples, that the damping ratio of a given aged soil in the field could be smaller than that of the corresponding sample reconstituted in the laboratory, measured under the same conditions. In this respect I believe that there is sufficient experimental evidence in the literature to support that idea (e.g. Watabe et al. 1991).

Next I would like to comment on the results presented in Fig. 56 (a) and (b) to link the liquefaction strength of dense and loose Toyoura sand samples to any one of the two following deformation properties measured in the laboratory: the maximum contractancy in drained triaxial compression tests (see Fig. 56 a) and the value E_{tan} defined for a shear stress level q at half of the peak strength q_{max} (see Fig. 56 b). Although there are only a few data points plotted in that figure, I would like first to pay attention to the fact that, when considering data points of loose and dense samples separately, the correlation between liquefaction strength and maximum contractancy given by the straight line in Fig. 56 (a) holds for both type of data. On the other hand the correlation between liquefaction strength and E_{tan} given by the straight line in Fig. 56(b) holds for dense sand and becomes poor for loose sand. Bearing in mind a) the important role played by the critical state or phase transformation lines in the liquefaction of loose sand and the cyclic mobility of dense sand (Sagaseta et al. 1991) and b) that critical states in triaxial compression tests are defined by points associated to maximum contractancy in volume-axial strain curves and to stress levels half the peak values in deviator-axial strain curves of dense sand (Luong, 1980) and by points associated with the largest contractive volume-strain attained at the maximum stress level in volume-axial strain curves of loose sand, the observations previously made concerning the correlations given in Figs. 56 (a) and (b) make sense. Consequently it can be concluded that the two straight lines in Fig. 5 represent the same failure criterion, valid for dense and loose sand in (a) but only for dense sand in (b).

The conclusions reached by the authors summarize the progess made in the field of laboratory testing at very small to small strains over the last 5 years. From the point of view of their influence on the day-to-day practice of geotechnical engineering they could perhaps be categorised into three types of news: bad, good and very good news. For example it is bad news to accept that we must modify our code of practice in laboratory testing to accurately reproduce the in situ behaviour of hard soils and soft rocks. It is also bad news to learn that the liquefaction potential of a given sand deposit is difficult to estimate based solely on field shear wave velocity, since that will restrict the use of liquefaction criteria based on seismic waves in seismic codes. On the other hand it is good news to learn that a continuous stress-strain relation for strain ranging from 0.001% to several % can be obtained from a single test using a single specimen. Finally it is very good news the confirmation that as the stiffness of a wide range of geomaterials becomes more strain rate-independent as the strain level decreases, an elastic threshold may be assumed to exist at a strain of 0.001%.

REFERENCES

LUONG, M.P. (1980), "Stress-strain aspects of cohesionless soils under cyclic and transient loading", Proc. Int. Symp. on Soils Under Cyclic and Transient Loading, Swansea, 1, 315-24.

SAGASETA et al. (1991), "Cyclic loading", X ECSMFE on Deformation of Soils and Displacements of Structures", Firenze, pp.: 981-99.

WATABE et al. (1991), "Large scale field tests on quaternary sand and gravel deposits for seismic siting technology", 2nd. Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Mo, I, pp: 271-89

Discussion on state-of-the-art paper titled: "The Evolution of Geotechnical Earthquake Engineering Practice in North America: 1954 - 1994" by W.D. Liam Finn and al. (Paper No. SOA3)

By: Vlad Perlea, U.S. Army Corps of Engineers, Kansas City District.

Since publication of Chinese criteria in 1979, based on observations following some strong earthquake, it is common practice to consider some clayey silts or silty clays as susceptible to significant strength loss during the seismic action. There is, however, no available information (based on these observations) on the magnitude of loss and the seismic action level that can trigger this behavior.

In post-earthquake analyses it is recommended to assume that these cohesive soils behave like sensitive clay, so that only the residual strength can be mobilized. However, do clayey materials loose strength in the same way as cohessionless soils? Is it justified to consider that the earthquake shaking capable to liquefy sands triggers also the strength loss in clayey materials, even if the expected deformations are insignificant? On the other hand, the approach of steady state or residual strength of liquefied sands (instead of zero strength) reduces the conservatism in dealing with cohesionless soils, that induced so much damage in the past. Therefore, we are now in the position of considering some clayey materials in the foundation of embankment dams more dangerous than liquefiable sands, i.e. possess comparable or even less strength than liquefied sands.

The concept used in the analysis of Sardis dam is appropriate considering the high hazard of the structure and its proximity to the New Madrid epicentral zone. It is likely that many practicing engineers will follow the same procedure in different conditions from the probable seismic activity point of view. Should this procedure be used, however, if a high hazard dam is located in a low seismicity zone, even if the design earthquake would be strong enough to trigger liquefaction in sands?

Local seismic activity may induce high maximum accelerations at the ground surface but low magnitude, and consequently short duration. The cohesionless materials may reach the steady state or residual strength, but the deformations at the end of the seismic action may be insignificant. If the mechanism of loss of strength in clayey materials requires extensive deformations it may not be justified to use the residual strength in post-earthquake analyses (both deformation and static stability).

The economic consequences of considering some clayey soils potentially dangerous for the embankment dam stability may be enormous. Therefore, it is my opinion that it is extremely important to develop an empirical correlation in order to limit conditions when loss of strength of cohesive soils should be considered a possibility. For example, the available empirical correlations between Liquefaction Occurrence, Magnitude and Epicentral Distance, show that no liquefaction was observed when the earthquake magnitude was less than about 5.2 (no reference to the type of deposits is usually made, but these correlations considered mostly locations where sand was present). I am confident that the threshold value of magnitude for clayey materials is higher than for all deposit types.

Discussion on paper titled: "The Evolution of Geotechnical Earthquake Engineering Practice in North America: 1954-1994," by W. D. Liam Finn (Paper No. SOA3).

By: James P. Lee, Senior Consultant, Brown & Root.

Figure 2 in this paper provides the liquefaction assessment chart developed by Seed et al. (1986). This figure shows that, based on field performance data, the silt increases the resistance to liquefaction of sands. However, recent studies, Vaid 1994 and Koester 1994, indicated that fine contents may actually reduce the resistance to liquefaction of sands. Until this apparent discrepancy is resolved and a conclusion can be drawn on the effect of the fine content, should this chart be still utilized for liquefaction analysis? In addition, this chart should not be extrapolated to beyond the 35% of fine contents.

Discussion on State-of-the-Art Paper titled: "Liquefaction-Induced Lateral Ground Displacement," by T. Leslie Youd Paper No. SOA6

By: Joseph P. Koester, U.S. Army Engineer Waterways Experiment Station (CEWES-GG-H), 3909 Halls Ferry Road, Vicksburg, Mississippi, USA 39180-6199

The author presents an empirical technique for estimating lateral displacements that may occur at the ground surface in the free field accompanying liquefaction. The technique proposed treats free-face and sloping ground conditions separately, and the author adequately recognizes the uncertainties inherent in the procedure.

The technique is attractive in its simplicity, requiring the use of parameters that are readily determinable or that would likely be estimated as part of any earthquake response study. The author has compiled an extensive collection of field case history data on which he bases his predictive formulae; it will be interesting and valuable to the profession for the author to update his data base and methodology as subsequent field occurrence data on lateral spreading become available. The recent strong earthquake at Kobe, Japan will certainly extend the author's data range to include lateral spreads accompanying liquefaction of granular deposits in the immediate vicinity of the causative fault.

The underprediction of displacements at the Juvenile Hall site, where the suspected soil contained an average 59% fines, is worthy of emphasis. Many of the Kobe liquefaction sites involved soils containing substantial fines of unknown plasticity and experienced large lateral displacements, apparently without developing flow slides in many cases. The discussor has measured very low undrained residual shear strengths in laboratory specimens prepared from artificial mixtures of sand and fines of varying plasticity; in loose mixtures, shear strain potential may be essentially unlimited. Flow sliding is likely in these materials, in which case the empirical displacement estimation procedure is inappropriate, as stated by the author.

A couple of minor editorial comments are offered:

(1) the author's use of mixed units within the text accompanying the example soil profile in Figure 12 is a bit confusing, as is the labeling on the axes of the figure (specifically, the Standard Penetration Resistance should be labeled as $(N_1)_{60}$ as described in the text); and
(2) the example calculation problem states that the upper liquefiable layer is 3.6 m thick, but the computations use 3.7 m. Neither of these situations detract from the overall high quality of the paper.

The author provides several examples where the application of the technique was trusted to support specific design decisions or to be regionally published for similar use. Given the constraints placed by the author on the use of the technique and the population of supportive data on which the regression expressions are based, engineers are offered a promising tool for non-critical design. The technique should be used to evaluate the efficacy of remediation when subsurface liquefaction potential is indicated and displacements on gently sloping ground might disrupt the function of a facility.

Discussion on paper titled: "**Recent Advances in Centrifuge Modelling of Seismic shaking**", by **Bruce L Kutter, University of California, Davis.** Paper No. SOA8

By: S.P.Gopal Madabhushi, Department of Engineering, University of Cambridge, England.

The author presents an excellent state-of-the-art review of some of the important aspects of modelling seismic events using a geotechnical centrifuge. In particular, the type of base shaking to which the centrifuge models are subjected, and the model containers which simulate the semi-infinite half space of the soil layers in the field, were emphasised. In dynamic centrifuge modelling we have progressed steadily from the initial euphoria of observing the liquefaction induced failures in our centrifuge models into an era of self evaluation of our modelling techniques and scaling laws. Several researchers are placing emphasis on the particle size effects, on the use of alternative pore fluids to avoid conflict in time scale factor (consolidation time Vs dynamic time period), on the effect of using high viscosity pore fluids on the dynamic properties of the soil especially damping, on strain rate effects etc. The author's efforts in including these topics must be appreciated. In this discussion we shall consider some of these aspects in some detail.

SHAKERS AND IMPORTANCE OF FREQUENCY CONTENT

There is no doubt that single frequency actuators like the bumpy road system (Kutter, 1982) and rotating cam system (Kimura et al, 1988), have helped in understanding some of the basic mechanisms of excess pore pressure generation and resulting degradation in soil stiffness (Steedman, 1991), and importance of initial natural frequency of soil-structure systems, (Madabhushi and Schofield, 1993). The author, however, comments on the importance of using multiple frequency base motions in centrifuge modelling. Based on the data presented in Fig.2 he argues that use of single frequency sinusoidal time history may sometimes lead to misleading conclusions. The data presented in Fig.2 refers to an experiment with a horizontal layer of Nevada sand over which a non-plastic silt layer is present. When a single frequency base motion was used the accelerometer in the middle Nevada sand layer exhibits extremely 'spiky acceleration' while the acceleration in the top silt layer die down after first few cycles presumably following generation of excess pore pressures. When a 'realistic' base motion was used the acceleration in the Nevada sand layer has less pronounced spikes. However, the time histories are not accompanied by the corresponding frequency analyses. The point made by the author will be more powerful when he can demonstrate that the single frequency input has caused significantly more amplification in Nevada sand layer at one frequency namely the base motion frequency. It would be of interest to know if the corresponding frequency component was amplified when a realistic base motion was used. This would confirm that amplification is a true phenomenon, and rule out the possibility of faulty instrumentation.

This also raises a much more important question on what is the ideal base motion that must be adopted in dynamic centrifuge testing. Earthquake engineers generally would agree with the author that use of a single frequency input may be inadequate to enable us a complete understanding of dynamic soil behaviour. However, the question that must be asked is 'whether to input all the frequencies at the same time by adopting some standard realistic earthquake' or 'to use a number of single frequency wave trains each with a slightly different frequency but spanning the entire range of frequencies of interest' (1~ 5 Hz for earthquake events). The later technique will have the advantage of providing an academic understanding of soil behaviour over the entire range of frequencies using the knowledge and techniques developed hither to for analysing data from bumpy road type base motions. In an academic examination of this matter it may be argued that use of a single 'realistic earthquake' type base motion will have the pitfall of not having the right frequency content of an earthquake that might strike in future, and hence not including some of the frequencies which could cause damage. At Cambridge University a new Stored Angular Momentum (SAM) based earthquake actuator is currently under manufacture which can fire strong earthquakes at a frequency of choice. For the research students at Cambridge it will be possible to sweep a range of frequencies using a 'micro-tremor' to detect the eigen frequencies. Following this strong earthquakes may be fired at an eigen frequency and the student may observe the damage sustained by the soil-structure system. This procedure will be explained in more detail in future (Madabhushi and Schofield, 1995). Initial numerical analyses have shown that a swept sine wave type input motion would be able to detect the natural frequency of soil-structure systems. As an example, let us consider the case of a horizontal soil bed which may be discretised as shown in Fig.D-1. The results from a finite element analysis in which a swept sine wave is applied at all the base nodes are presented in Fig.D-2. In this figure the acceleration-time histories at different heights along the middle of the soil bed are shown. As the swept sine wave propagates vertically upwards to the sand surface, amplification is observed at two distinct frequencies which correspond to the natural frequencies of the soil bed. This shows the suitability of swept sine wave type base motions which can act like a micro-tremor and may be used to identify the eigen frequencies. All being well it is possible that SAM based earthquake actuators can be designed and built to fire strong earthquakes on deep foundation models in very high gravity centrifuge tests of up to '300g'.

MODEL CONTAINERS

The author emphasises the need to consider and evaluate all the aspects of centrifuge model containers in simulating the semi-infinite half space. The options available at the moment are to use laminar boxes which allow lateral deformation or to use a boundary with the same dynamic stiffness as the soil body (Equivalent Shear Beam or ESB containers). In both the cases the boundary must generate complementary shear stresses. It is important to evaluate the performance of the thin, inextensible shear sheets which provide the complementary shear stresses in both type of model containers. Madabhushi, Schofield and Zeng (1994) carried out a study on evaluating the performance of the shear sheets when used with duxseal boundary. Work is underway at Cambridge University to conduct similar study when shear sheets are used with the ESB model containers. Also in future sophisticated laminar boxes may be designed with very light yet very strong Carbon fibre reinforced materials. This will bring down the ratio of mass of boundary to that of the soil body thus reducing the interaction between the boundary and the soil body. In all this the centrifuge modeller must keep in mind the phenomena being investigated. If a soil-structure system is being studied with probable near field liquefaction but without the far field being liquefied then the ESB container models the half space satisfactorily. If fluidisation of the far field is of importance, then model containers with very light end walls which allow lateral deformation of soils may be more suitable. The author's comments on considering the interaction of shaker-container-soil body are particularly useful for the numerical modellers attempting to analyse centrifuge test data.

TIME SCALE FACTOR CONFLICT

The scaling of time in diffusion processes like consolidation ($1/n^2$) differ from the scaling of time in dynamic events ($1/n$). The options available to avoid this conflict in time scale factors are either to use reduced size particles so that the permeability is reduced, or to use a model pore fluid of high viscosity. Use of reduced size particles may be equivalent to using soil with different mechanical properties. This may lead us into the same discussion as the particle size effects, and the author's comments in that section apply. Also, use of model pore fluids with increased viscosity must be evaluated carefully. In particular the effect of the model pore fluid on the dynamic properties of the saturated soil body such as damping must be considered carefully. A comparison of damping associated with tower structure-soil systems with water and silicone oil as pore fluids was carried out by Madabhushi (1994). Similar studies are to be undertaken with other model pore fluids made of Methyl Cellulose, Glycerine etc. Also, future research must concentrate on determining the strain rate effects in dynamic centrifuge tests.

VELACS PROJECT

Verification of Liquefaction Analysis by Centrifuge Studies (VELACS) was the first project with a major collaboration between seven universities each with dynamic centrifuge test capabilities. While much of the significant behaviour was reproduced, by and large in all model tests at different venues, the VELACS project helped in identifying some of the difficulties in fully reproducing the centrifuge tests. While the difficulty in reproducing the base motion was highlighted, this project did help in creating a large experimental data base on specific experiments for validation of new numerical procedures.

REFERENCES

Kimura,T., Takemura,J. and Saitoh,K., (1988), Development of a Simple Mechanical Shaker using Cam Shaft, Centrifuge 88, Paris, Corte (ed.), Balkema, Rotterdam, pp.107-110.

Kutter, B.L.,(1982),Centrifugal modelling of the response of clay embankments to earthquakes, Ph.D. thesis, Cambridge University, England.

Madabhushi, S.P.G., (1994), Effect of pore fluid in Dynamic Centrifuge Modelling, Centrifuge 94, Leung, Lee and Tan (eds.), Balkema, Rotterdam, pp 127-132.

Madabhushi, S.P.G. and Schofield, A.N.,(1993), Centrifuge modelling of tower structures on saturated sands subjected to earthquake perturbations, Geotechnique Vol.43, No.4.

Madabhushi, S.P.G. and Schofield, A.N.,(1995), Simulation of earthquakes in a geotechnical centrifuge, IS-Tokyo'95, First Intl. Conf. on Earthquake Geotechnical Engineering, 14-16 Nov., Tokyo, Japan.

Madabhushi, S.P.G., Schofield, A.N. and Zeng, X., (1994), Complementary shear stresses in dynamic centrifuge modelling, Dynamic Geotechnical Testing II, ASTM STP 1213, R.J.Ebelhar,V.P.Drnevich and B.L.Kutter (eds.), American Society for Testing and Materials, Philadelphia.

Steedman,R.S.,(1991), Centrifuge modelling for dynamic geotechnical studies, Proc.II.Intl.Conf.on Rec.Adv.in Geo. and earthquake eng., St.Louis, USA.

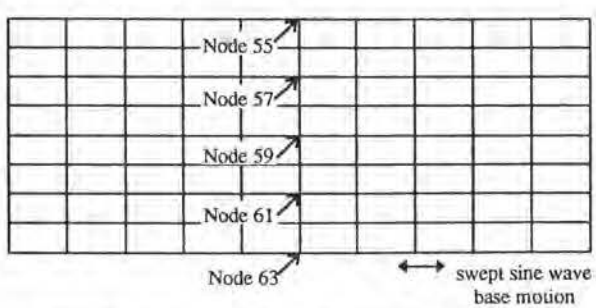

Fig.D-1. Schematic diagram showing the F.E. discretisation of the horizontal sand bed

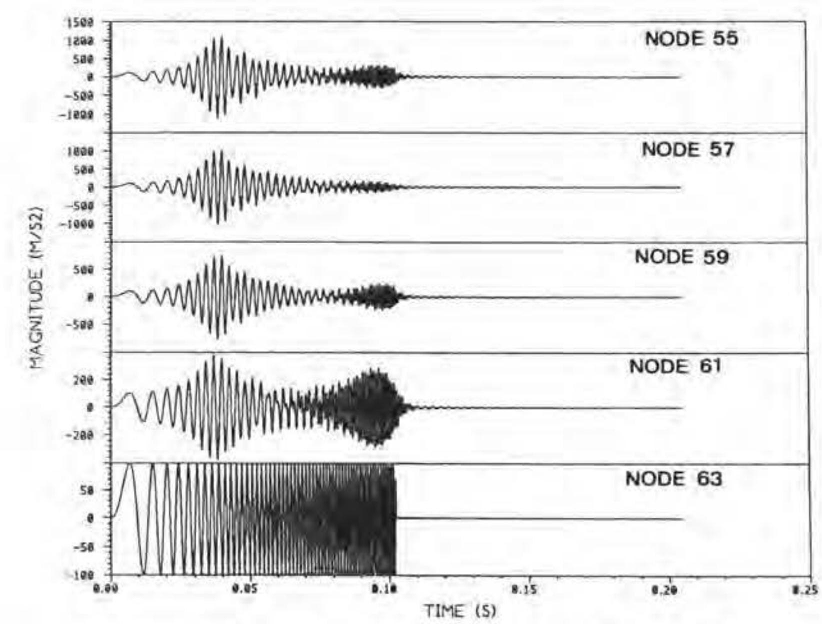

Fig.D-2 Acceleration-time histories at different nodes in the horizontal sand bed

Discussion of "Recent Advances in Centrifuge Modeling of Seismic Shaking"

Hubert K. Law
Dept. of Civil Engineering
University of Colorado, Boulder

The author is to be congratulated for his efforts in presenting the state of the art of earthquake modeling in geotechnical centrifuges. The paper includes four main issues, namely (1) the need for shaking system in a centrifuge which can deliver virtually any earthquake motion and development of these shakers, (2) discussion on model containers and their boundary effects in terms of both experimental and analytical stand points (3) scaling relations pertaining to dynamic testing in a centrifuge, and (4) brief review of VELACS research project. The writer would like to discuss some of the materials presented in the paper and other important issues.

IMPORTANCE OF FREQUENCY CONTENTS

Aside from some shake tables limitations to produce multi-frequency base motions, many researchers continue to use single frequency sinusoidal waves to excite their models for simplicity. The author pointed out that such base motions were not realistic to represent actual earthquakes and might provide a false impression. The writer agrees with his comments, if they use the finding of geotechnical structures based on a single frequency excitation to generalize typical earthquake behaviors, but the usage of single harmonic has its own merit. It is important that researchers clearly understand soil-structure behaviors under a simple excitation mode as a first step toward understanding of a more complicated excitation. As the understanding of soil behavior under a single harmonic excitation toward an earthquake excitation is a big step, the usage of single harmonic motions should not be discouraged. The author claimed the importance of frequency by showing a different character of sand layer excited by an uniform sinusoidal motion from an earthquake-like motion. As the nature of the two input motions are completely different, i.e., both frequency contents and amplitudes, it would be rather difficult to compare the soil responses excited by these base motions. One cannot distinguish whether the difference is due to frequency or due to amplitude. Although the frequency-dependent nature of soil is the fact like any other dynamically excited body, the answer will be more obvious if results of ground response subjected to several single-harmonic base motions with different frequencies but the same amplitude are available.

SHAKERS

The author presented a multi-actuator shaking system concept and design for a centrifuge. The concept has been commonly shared by researchers in the University of California, Davis, Rensselaer Polytechnic Institute, and the University of Colorado, Boulder. Having experience from a smaller servo-hydraulic shaking system, they seem to recognize the need to minimize uncontrollable rocking motions brought about by placing the actuator underneath the table which creates a large moment arm on the model container. The need has been implemented in the current design of their large shake tables by reducing the distance between the actuator's force application point and the center of payload mass. As more than one actuator is pushing the table in one direction, there is inevitable interaction between actuators. As a result of being unable to synchronize, some hydraulic energy will be wasted when one actuator fights another. Thus some form of control algorithm is useful not only to take into account for the cross-talk between them but also to deliver a specific motion of the shake table with a reasonable accuracy. There is a control algorithm for vibration tests (Favour, 1974), and this idea has been implemented in a centrifuge on a single actuator shaking system (Ketcham, et al., 1991). The principle is based on the wave form control algorithm, which uses the measured transfer function of the shaking system to predict the command signal needed to produce a desired shake table motion. The difference between the measured and desired motions is systematically reduced. To carry out this kind of correction algorithm, a shake table's transfer function has to be determined experimentally to account for any nonlinearity. This algorithm, already developed for a single actuator, can be extended to a generalized form for dealing with multi-actuator situations. In this case, the challenge will be the physical determination of transfer matrix $[H_{ij}(\omega)]$ which is no longer a scalar quantity $[H(\omega)]$ as in the case of single-actuator system. $H_{ij}(\omega)$ is for the response of actuator i due to unity input at the actuator channel j for an angular frequency ω.

BOUNDARY EFFECTS FOR LEVEL GROUND MODELS

Both the author and Ko (1994) discussed issues relating to container end boundary effects. Laminar containers, stacked ring apparatus, hinged plate containers (HPC), equivalent shear beam containers (ESB), and flexible shear beam containers (FSB) have been developed for simulating a level ground. The numbers of rectangular rings employed in these containers range from 4 to 48. As it is not clear whether 4 rings are sufficient to compose a container, there is a need to investigate the minimum acceptable number of rings to be used which may depend on the type of application. Intuitively, the container with a large number of rings would have a better chance of simulating a continuous soil column. The HPC offers the ability to rotate the end plates to provide continuous displacements at the end boundaries; this condition that is not achieved in laminar containers is desirable. Also, unlike most laminar containers, each plate of HPC is supported individually, and thus its weight will not be transferred to the lower layers. This design significantly reduces the frictions on the lower rings. However, all the laminated containers will suffer step displacements on the side boundaries due to a discrete nature of the rings. The problem may be severe when the container is relatively narrow and is composed of only a few rings. A liquified soil layer, which by definition has zero shear stiffness, can deform grossly and thus this condition must be accommodated by the container. The ideal container thus will have zero shear stiffness and provision of large deformations. In practice even a laminar container or HPC, which is constructed with bearings, has finite shear stiffness due to small frictional resistance of bearings (coefficient of friction is typically 0.005 to 0.01), but they can provide large deformations. The ESB container using rubber sheets instead of bearings to match the container natural frequency to that of soil layer may not be suitable for liquefaction studies as the author indicated that the natural frequency of the soil reduces when softening occurs. As a result, the shear stiffness of the container becomes larger than the soil layer. The improvement is made in the FSB container. When a container relies on stretching of the rubber sheets, it seems unable to provide large deformations which are likely to occur in a soil layer during liquefaction unless the rubber sheets are thick and quite flexible. Although significant efforts, including the recent implementation of complementary shear stresses in HPC, ESB, and FSB boxes, have been made to meet the design criteria for an ideal container proposed by Whitman and Lambe (1986), the final product of the container has to be acceptance tested. Since the ultimate goal is to simulate a soil layer of infinite extent which yields identical motions along a same elevation when its base is excited by an earthquake, the motion uniformity in the container should be checked to verify the effectiveness of container's design.

BOUNDARY EFFECTS FOR NON-LEVEL GROUND MODELS

For an embankment model where both ends of the slope do not touch the container, it may be feasible to use a rigid container. If a plane strain condition is to be preserved, the container would have free friction on side boundaries and no deflection in the outward directions. The friction may be reduced by applying grease and a layer of latex sheet. But when the embankment is to retain a reservoir on one side and to establish seepage flows in it, the greased wall is not capable of preventing flow of water along the embankment-container side interface. It may be better not to lubricate the side boundaries in this case. However, soil arching problems due to the friction of unlubricated side boundaries can be significant especially the container's aspect ratio (width/height) is relatively small. In fact soil arching is not unique to the embankment problems; it can effect any plane strain model if boundaries are not treated properly. In addition to the problem of side boundaries, end boundaries of a retaining wall are of special concerned since the soil layers on the both sides of retaining wall are different in thickness; they contact the container's end walls. As indicated by Ko (1994), it is not clear in regard to what type of container is suitable for retaining wall problems.

ANALYSES FOR CENTRIFUGE MODELS

The author suggested analyzing dynamic centrifuge test results by incorporating the interactions of container, shake table, and the centrifuge reaction mass as a rigorous approach. If the actuator displacements relative to the centrifuge bucket is the only input motions available to analysts, the interaction of reaction frame including the centrifuge structure should be included in analysis. On the other hand, if the absolute base motions of the model with respect to an inertial reference is available, the effects of actuator, reaction mass, and centrifuge structures can be ignored. In fact, base acceleration measurements provide such absolute motions. In solving a boundary value problem, either a displacement (absolute) or traction field must be prescribed to the boundary. Therefore the geotechnical structure under studied can be isolated from the shake table and centrifuge, and the solution can be obtained by prescribing appropriate boundary conditions. If the container base is relatively stiff, its motions can be used to prescribe the bottom boundary of the model, but due to flexibility of the container sides, the interaction between the container and soil should be taken into account.

INSTRUMENTATION AND DATA ACQUISITION

Instrumentation and data acquisition issues are not discussed in either the author's or Ko's (1994) paper. These are one of the more important components in dynamic centrifuge testing. With advances in development of sophisticated shaking system, the status of data acquisition follows closely. Unfortunately, the matter is not widely discussed. Perhaps researchers should present and discus the status of their systems and learn from each other so that new ideas can be shared as well as some potential pitfalls can be avoided.

It is quite common, at least in small centrifuges, to bring analog signals (conditioned or unconditioned) from electrical slip rings and to digitize them outside of centrifuge. The electrical slip rings often introduce noises in these signals, and the problems may be serious for unconditioned low voltage signals. Some use fiber-optic slip rings rather than conventional electrical ones (Nagura, et al., 1994), and they find the system to be satisfactory. One can also use telemetry for wireless transmission, but, to the writer's knowledge, this idea has not been implemented in a centrifuge. Perhaps a better system will be placing the entire data acquisition system on the centrifuge to condition and digitize before going through slip rings. In this case only digital data which is less susceptible to noise will be transmitted through electrical slip rings. The on-board data acquisition can be operated remotely through some forms of communications, and the ethernet communication has been found to be effective (Kutter and Lakeland, 1994). A good data acquisition system should have an analog filter, amplifier, and ability to offset signal for each channel. The combination of amplifier and offsetting capabilities should give a maximum resolution, while analog filter will prevent an aliasing problem. As aliasing is a serious problem not only in dynamic testing but also in static testing, researchers need to pay close attention on this issue. Once the data is aliased, there is no way of removing the effects. Aside from types of data acquisition, it is also important that an experimentalist chooses not only the right kinds of transducers but also right operation ranges for different types of applications. Certainly, some knowledge of transducer construction will be helpful.

ACKNOWLEDGEMENT

Discussions with Dr. B. Kutter and Mr. J. Lakeland of UC-Davis, and Drs. E. Stauffer and H. Ko of CU-Boulder are acknowledged..

REFERENCE

Favour, J. D. (1974), "Transient Waveform Control, A Review of Current Techniques," *Journal of Environmental Science*, Vol. 17, pp 9-19

Ketcham, S., H. Ko, and S. Sture (1991) "Performance of an Earthquake Motion Simulator for a Small Geotechnical Centrifuge," *Centrifuge 91*, Ko and McLean (eds.), Balkema, pp 361-368

Ko, H. Y. (1994), "Modeling Seismic Problems in Centrifuges," *Centrifuge 94*, Leung, Lee and Tan (eds.), Balkema, pp 3-12

Kutter, B. L., and J. Lakeland (1994), Private Communications

Nagura, K., M. Tanaka, , K. Kawasaki, Y. Higuchi (1994),"Development of an Earthquake Simulator for TAISEI Centrifuge," *Centrifuge 94*, Leung, Lee and Tan (eds.), Balkema, pp 151-156

Whitman, R. V., and P. C. Lambe (1986), "Effect of Boundary Conditions upon Centrifuge Experiments Using Ground Motion Simulation," *Geotechnical Testing Journal*, Vol 9, No 2, pp 61-71

Discussion on paper titled: "Simple Physical Models for Foundation Dynamics", by John P. Wolf, State of the Art Paper No. 9.

By: Fabrizio Pelli, Consulting Engineer, Bogliasco (Genoa), Italy.

Dr. Wolf's paper is an important contribution to the state of the art on dynamic modelling. The following considerations are partly related to the analytical aspects of the paper, and partly to the potential applications in design.

a) The writer shares the author's opinion that the approximations of the simplified methods are generally acceptable, in consideration of other uncertainties related for instance to soil properties (and seismic input). However, the user should always make sure that the applied method is suitable for specific design conditions. As an example, the comparison between the dynamic stiffness coefficient for a disk on a two layer system (stiff over soft layer) shown in Figure 3, indicates that the unfolded layer cone fits well the exact solution up to a normalised frequency value $a_0 \approx 6$. On the other hand, for higher frequencies the spring coefficient is underestimated considerably in this particular case. A similar behaviour is also observed in Figure 10c, for a soil layer on rigid rock, where at normalised frequencies exceeding 6 the approximation becomes poorer. This high frequency range is not relevant for most applications, and in particular for seismic shaking where a time-domain approach is most needed (due to the various frequency components of a typical earthquake motion). On the other hand, $a_0 > 6$ may have to be considered for some turbines with operating frequencies exceeding 80Hz. In this case, a simple spring+dashpot discrete model calibrated on exact solutions at the machine operating frequency seems appropriate. It appears that the basic lumped mass parameters given in Tables A-5 and A-7 are best suited for time-domain analyses at normalised frequency values not exceeding 6.

b) Section A1 deals with models for surface foundations on a homogeneous half space. The same case for embedded foundations is discussed in Section A3. The cone model applies to circular foundations, and non circular foundations must be somehow approximated. The relationship provided in Table A-1 (i.e. $r_0 = \sqrt{\frac{A_0}{\pi}}$) can be applied to foundations of different shape, provided that their aspect ratio is sufficiently low (say up to two or three). As an example, for these foundations a frequency independent horizontal stiffness (and damping) can be assumed. For higher aspect ratios the effects of frequency doesn't seem to be negligible (e.g. Gazetas, 1991). Therefore a frequency dependent spring stiffness (not suitable for analysis in the time-domain), or an appropriate lumped-parameter model could be adopted to improve accuracy. A similar consideration can be made for the other stiffness and damping components, including the vertical stiffness whose variability with frequency is also affected by the foundation aspect ratio. It appears that the lumped-mass parameters given in Tables A-2, A-3 and A-4 are best suited for time-domain analyses of foundations characterised by a relatively low aspect ratio.

c) In Sections A2 and A3 surface and embedded foundations in a layer resting on rigid rock are discussed, respectively, and a lumped-parameter model with frequency independent stiffness, damping and mass parameters is defined (Figure A-12). The model provides a remarkably good approximation of the exact solution (Figure 10f), at least for a_0 values lower than 6. Moreover, the geometric damping becomes very low and gradually vanishes when the excitation frequency becomes lower than the first natural frequency of the soil layer. Due to its ability of approximating the interaction impedance at low frequency levels, this model appears to be most suitable for time-domain seismic analyses, particularly for flexible structure-foundation systems.

d) The lumped parameter model discussed at Point c) appears particularly attractive in view of an extension to cases where the impedance contrast between the soil layer and the bedrock is limited (i.e. stiff soil on fractured/weak rock), and where the soil-on-rigid-rock assumption may lead to overly conservative damping values at low frequencies.

e) The simplified methods discussed in the paper can provide a simple means to investigate soil softening associated with cyclic loading. For instance, the solution of the soil layer with elastic rock subgrade appears suitable to model in a simplified manner the effects of earthquake-induced cyclic mobility of the top layer on the soil-structure interaction parameters. A linear equivalent approach can be used, by defining for each soil layer a set of secant dynamic properties. The input parameters can be estimated by free field site response analyses.

f) Near field effects could also be taken into account in a simplified manner, if softening of a thin horizontal layer can reasonably be assumed. A non linear spring (e.g. Nogami et al., 1992) or a linear-equivalent spring+dashpot model and consistent masses could be placed in series with the model outlined at Point e). If a linear equivalent approach is selected, iterations are required as the stiffness of the top layer and the seismic response of the structure-foundation system affect each other. The thickness and stiffness of the top layer could be evaluated at each iteration by a simple static-elastic approach, and in light of proper information on soil behaviour.

g) The approaches e) and f), although highly simplified, could provide useful sensitivity evaluations, and could be easily implemented within a probabilistic framework.

REFERENCES

Gazetas, G., (1991), "Foundation Vibrations", in Foundation Engineering Handbook, 2nd ed., Fang editor, Van Nostrand Reinhold, pp.553-993.

Nogami, T., J. Otani, K. Konagai and H. Chen, 1992, "Non linear Soil-Pile Interaction for Dynamic Lateral Motion", J. Geotech. Engrg., ASCE, Vol. 118, No. 1, pp. 89-105.

Discussion on
"Simple Physical Models for Foundation Dynamics"
by John P.Wolf (Switzerland), Paper #SOA9

By Maotian Luan and Gao Lin
Professor, Department of Civil Engineering,
Dalian University of Technology,
Dalian 116024, P.R.China

The paper presented theoretically-meaningful and practically-valuable investigations on the physical modelling of foundation dynamics in various simplified procedures. Special emphases on the cone model and lumped-parameter model reproducing dynamic impedances of underlying foundations on footing and structures in soil-structure interaction analysis are systematically made by the author. The effort of the paper is oriented to provide a simple and feasible methodology on a rational theoretical background for considering the effect of subsoils on structural seismic response in engineering practice.

The cone model or wedge model respectively for 3- and 2-dimensional problems, pioneered by Meek and Veletsos (1974), gained popular extensions (e.g., Gazetas and Dobry 1984; Gazetas 1987; Dobry and Gazetas 1988), refinements and generalizations mainly by Wolf and Meek in recent years. The related problems including the followings should be further solved:

1. For sway, rocking and torsional vibration modes, the shear-wave velocity can be used since the major shear-wave propagation in soils is induced. For vertical oscillating, however, it may be difficult to choose a rational material modulus or medium wave velocity. It cannot be certain that only the dilatational-wave propagation is induced in soil media. At present, two options are recommended in practical usage, i.e., dilatational-wave velocity corresponding to the constrained modulus and the Lysmer's analog "wave velocity".

2. There is no better method for defining a reasonable values of static stiffness for the horizontal or vertical translational vibration modes used in dynamic impedance representations.

3. For the layered soils, the natural vibration characteristics of soil deposits may tend to yield the heavy or sharp oscillation phenomena of dynamic stiffness (Takemiya, Luan and Lin 1990). Special improvements should be conducted in the extension of the cone model to layered non-homogenous foundations. In these cases, simplicity of the method may be balanced by its complexity.

The lumped-parameters models are widely employed in representing the dynamic impedance functions of elastic or visco-elastic half-space foundation due to its simplicity (Gazetas 1983). It is more and more recognized that these models make the time-domain analyses for nonlinear soil-structure interaction problems feasible. Therefore concentrated attempts in this direction are performed. The simple 1-degree-of-freedom model with two mechanical elements consisting of a spring and a dashpot cannot gain a fairly accurate results. Multiple-DOF models with a number of parameters typically as shown in Fig.1 (e.g., De Barros and Luco 1990; Jean, Lin and Penzien 1990; Luan, Lin and Chen 1995) are developed recently. The increase of degrees-of-freedom of such discrete element models will enhance the sophistication in determining the optimization values of the related parameters. On the other hand, it will raise the scale and amount of computational work for soil-structure interaction coupling systems. Therefore it is proposed that the number of discrete elements should be limited, for example, 2- or 3-DOF systems with 2 to 10 parameters. In this way, the simple physical elements should be combined together in such an optimized manner that the defined lumped-parameters model can gain a best accuracy for fitting the actual dynamic impedances and reduce the least computational effort in coupling analysis. It should be noted that some of springs or dampers obtained by mathematical optimization procedures may take negative values which lose their physical nature or meaning.

De Barros, F.C.P. and J.E. Luco (1990), "Discrete Models for Vertical Vibration of Surface and Embedded Foundations", Earthquake Engineering and Structural Dynamics, 19:281-303.

Gazetas, G. (1983), "Analysis of Machine Foundation Vibrations: State of the Art", Soil Dynamics and Earthquake Engineering, 2(1):2-42.

Jean, W.-Y., T.-W. Lin and J. Penzien (1990), "System Parameters of Soil Foundations for Domain Dynamic Analysis", Earthquake Engineering and Structural Dynamics, 19:541-553.

Luan, M.T., G. Lin and W. F. Chen (1995), "Lumped-Parameter Model and Nonlinear DSSI Analysis", Proceeding of the 3rd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis.

Takemiya, H., M.-T. Luan and G. Lin (1990), "Simplified Approach to Dynamic Analysis of Foundations on Non-Homogeneous Soils", Proceedings of the 8th Symposium of Japanese Earthquake Engineering, 1:319-324.

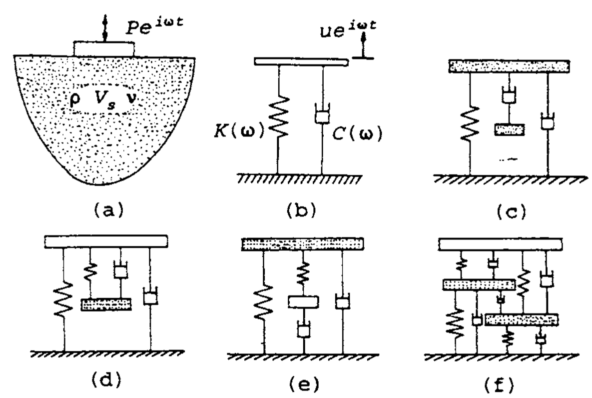

Fig.1. Footing on Elastic Half-Space and Various Discrete Models

Discussion on State-of-the-Art paper titled "Clay Behavior, Ground Response and Soil Structure Interaction Studies in Mexico City", by Dr. Miguel P. Romo (Paper No. 13)

By: Madan B. Karkee, Development Division, GEOTOP Corporation, Tokyo, Japan and Yoshihiro Sugimura, Dept. of Architecture, Tohoku University, Sendai, Japan

Dynamic behavior of Mexico valley clay has been a subject of keen interest to geotechnical earthquake engineers and seismologists worldwide. The writers would like to note some of the points mentioned in the paper that may be further clarified for better understanding of the sate of the research, and would also like to present examples of dynamic behavior of some clay types encountered in Japan for general comparison and discussion.

STRAIN-DEPENDENT DYNAMIC PROPERTIES

Compared to well known clay types from other parts of the world, the Mexico valley clay has long been well known for its practically elastic behavior up to fairly high strain levels (Romo *et al*, 1988; Seed *et al*, 1988). The author presents a rational approach to simulating this unique clay behavior considering its dependence on the plasticity index I_p and the relative consistency I_r.

The strain at which the shear modulus begins to degrade appreciably is referred to as the threshold strain and its value for Mexico clay is reported to vary from 0.2 to 0.5% depending on the value of $(I_p - I_r)$. I_p is stated to depend on the initial microstructures of soil and not to be so much dependent on the physical and chemical changes that may have occurred since its formation. I_r is considered to account for any modifications over time in the initial microstructures of soil. However, such correspondence may not be easily evident and further elaboration in this regard would be beneficial. Also, if $(I_p - I_r)$ is more representative of the shear modulus degradation characteristics, presumably it may be reasonable to expect the soil parameters γ_t, A and B to be functions of $(I_p - I_r)$ instead of I_p.

Ohsaki, Hara and Kiyota (1978) have presented an expression for average skeleton curve given by Equation 1 for modelling nonlinear response of soils in Japan. Using Masing's type model, the damping ratio is given by Equation 2. Here, τ is the shear stress and S_u is the shear strength. The soil parameters $\alpha = 0.01 \times G_{max}/S_u - 1.0$ and β is 1.4 for clay and 1.6 for sand.

$$\frac{G}{G_{max}} = \frac{1}{\left\{1 - \frac{1}{1+\alpha\left|\frac{\tau}{S_u}\right|^{\beta}}\right\}} \tag{1}$$

$$\lambda = \frac{2}{\pi}\frac{\beta}{\beta+2}\left\{1 - \frac{1}{1+\alpha\left|\frac{\tau}{S_u}\right|^{\beta}}\right\} \tag{2}$$

Figure 1 shows comparison of experimental results for clay samples from Sendai (Karkee, 1993) with Equation 1 considering G_{max}/S_u of 600 and $\beta = 1.4$, values recommended for clay. Fairly good agreement can be noted. However, Equation 2 is seen to significantly overestimate the damping values for clay samples in Figure 2. When the damping ratio is estimated by the author's approach with λ_{max} and λ_{min} assumed to be 14% and 3% respectively (Equation 3), the data is represented quite well in Figure 2. The author's approach seems effective in ensuring that the damping at any level of shear strain during nonlinear response analysis does not exceed realistic values based on test results.

$$\lambda = 14.0 \times \left\{1 - \frac{G}{G_{max}}\right\} + 3.0 \tag{3}$$

There is a rather unique type of soil encountered in Japan that is known as the peat. It has high I_p and is found to behave elastically up to quite high strain levels. As a result, its dynamic behavior cannot be represented adequately by Equation 1.

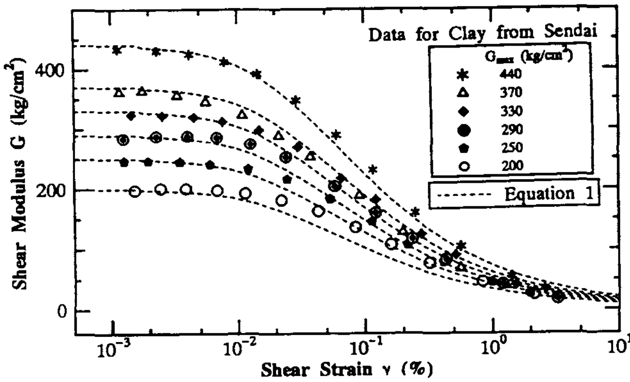

Figure 1: Comparison of data for clay from Japan with Equation 1.

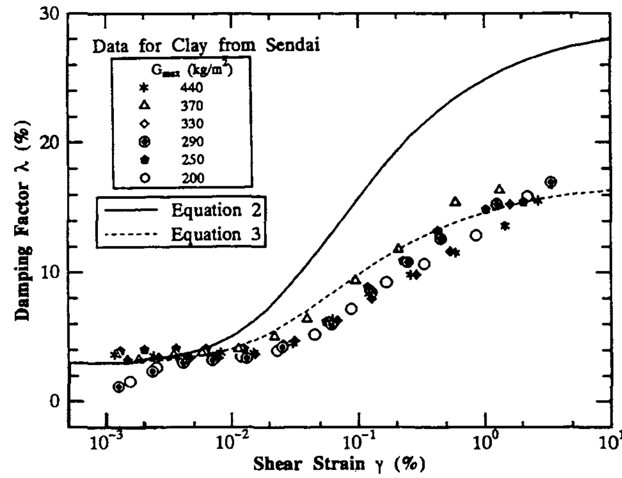

Figure 2: Clay damping data from Japan and Equations 2 and 3.

Figure 3 shows the shape of $G/G_{max} \sim \gamma$ curves for five of the soil samples from Mexico city tabulated in the paper. Values of the soil parameters γ_t, A and B of the author's model are approximated from relevant figures in the paper. The corresponding values of $(I_p - I_r)$ are shown. The experimental data for peat from Japan are plotted with circles in Figure 3. From this approximate plot, it seems difficult to see a direct correspondence between $(I_p - I_r)$ and the so called threshold strain. The shear modulus degradation behavior of peat, however, shows good resemblance with that of Mexico city clay.

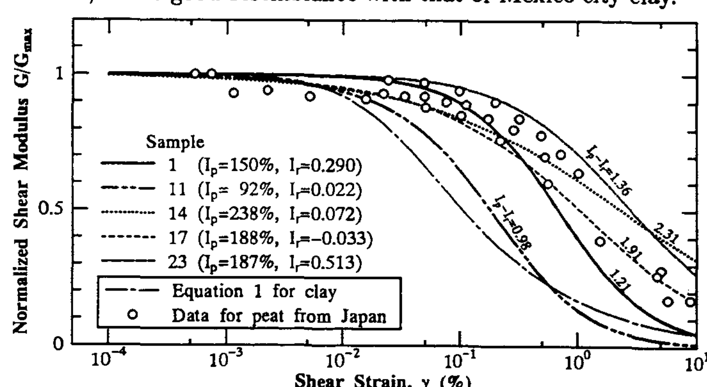

Figure 3: Author's model for Mexico clay samples and data for peat.

One of the five samples (11) shown in Figure 3 has comparatively low I_p and, consequently, indicates a threshold strain of about 0.01%. The curve given by Equation 1 for clayey soils in Japan is also plotted in Figure 3 for comparison and its general trend is seen to be quite similar to that of sample 11. It would be interesting to know how often the soil deposits with I_p less than about 100% are encountered in Mexico valley, in which case the strain level at which the shear modulus begins to degrade substantially seems likely to be quite low. Considering that only one of the 23 samples tabulated in the paper has I_p less than 100%, it may in fact be quite infrequent.

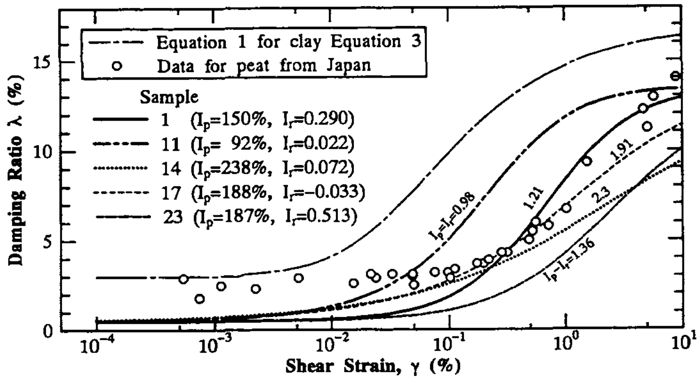

Figure 4: Damping model for Mexico clay samples and data for peat.

Figure 4 shows the variation of damping ratio for the corresponding cases shown in Figure 3. Values of λ_{max} and λ_{min} for Mexico clay are 13% and 0.5% as suggested by the author. Again, it may be necessary to reconfirm whether these values are adequate for the soil sample 11. The damping data for the peat denoted by circles in Figure 4 show λ_{min} of about 2%, well above that reported by the author for Mexico city clay. However, the trend of increase in λ with shear strain is quite similar. Damping for clay given by Equations 1 and 3 is also shown in Figure 4 for comparison.

Figure 5 shows $G/Gmax$ and λ data for peat fitted with the author's model. Although the relevant data were not available, a few trials with the author's model showed that I_p and I_r of 200% and 0.05 respectively results in a reasonable fit to the data.

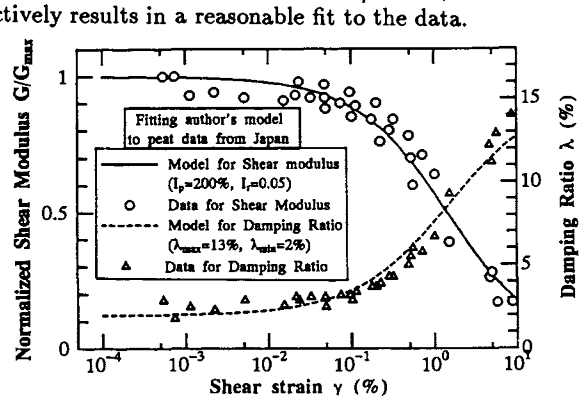

Figure 5: Representation of peat data from Japan by author's model.

Reporting very interesting results regarding the correlation between cyclic and permanent strains, the author has concluded that Mexico clay develops significant permanent deformation only when it is close to failure under dynamic loading. The typical clay sample, for which the correlation is given as an example, has $I_p = 250\%$. Further comment on whether permanent deformation characteristics would be same at lower value of I_p (say at $I_p = 150\%$) would be informative.

Previous publications on Mexico city soil conditions and ground behavior (Seed, Romo *et al*, 1988) indicate the existence of 4 to 5m thick layers near ground surface classified as sand and silty sand. No particular mention is made in this paper about the behavior of sandy soils. A brief comment in this regard would be very helpful.

GROUND RESPONSE IN MEXICO CITY

The drastic variability in the response spectra characteristics of the motions recorded during the 1985 event at different sites, with seemingly similar overall shear wave velocity profiles, is indeed remarkable. But even more noteworthy is the verification that the one dimensional modelling for response analysis can reproduce such variability very closely when the dynamic behavior of soil is adequately represented. Unique seismic response phenomenon observed in the Mexico valley has drawn very intense research interest and as a result several two dimensional models have been advanced in the past several years. The author presents a systematic discussion on these developments together with complexities and shortcomings involved, and also explains how one dimensional model was erroneously considered incapable of replicating some of the response features actually observed in Mexico city such as beating and long coda.

The one dimensional model for response analysis reported in the paper is in conformity with the observed ground response. However, no mention is made regarding the level of shear stress and shear strain actually attained during the response analysis of soil deposits. Such information can be indicative of the extent of nonlinearity in soil response. For example, vertical variation of motion in the lake zone is noted to be amplified much more by upper 30m of the stratigraphy than in the deeper 72m. It concerns a small event motion. It would be interesting to see if the upper layers can be shown to exhibit similar amplification characteristics by carrying out response analysis of the soil deposit with a much stronger input motion at the base layer.

In one of the earlier publication (Romo and Seed, 1986), it is concluded that peak accelerations of surface response motion computed from one dimensional modelling at different sites correlated well with the depth of clay deposits, and that the regions with higher peak acceleration correspond well with the concentration of damage. In this paper the author has utilized the response spectra as the basis for comparison and discussion. It would be very informative to request author's comment on interrelationship, if any, between peak acceleration, depth of soft soil deposits and the observed damage patterns. It would also be interesting to know if the results of 1985 publication could be reproduced by author's current one dimensional model.

The author's case studies and theoretical studies regarding the soil structure interaction effects in Mexico city indicate subtle to drastic difference between the excitation incident on structures and the free field motion, depending on the type of foundation. It is reported that deep box type foundations attenuate the free field motions significantly compared to friction pile foundations. These examples and simulations seem to concern small event motions. A comment on whether similar distinctions can be expected in case of intense shaking would be helpful.

REFERENCES

Karkee, M. B. (1993), "Seismic Microzonation of Urban Areas considering the Levels of Excitation and the Local Soil Nonlinearity", Doctoral Dissertation, Dept. Architecture, Faculty of Engineering, Tohoku University, Sendai, Japan (117-119).

Ohsaki, Y., Hara, A. and Kiyota, Y. (1978), "Stress-strain model of soils for seismic analysis", Soils and Foundations, 13(4):61-73.

Romo, M. P., Jaime, A. and Reséndiz, D. (1988), "The Mexico earthquake of september 19, 1985 - general soil conditions and clay properties in the valley of Mexico", Earthquake Spectra, EERI, 4(4):731-752.

Romo, M. P. and Seed, H. B. (1986), "Analytical Modelling of Dynamic Soil Response in the Mexico Earthquake of Sept. 19, 1985", Proceedings, ASCE Conference on Mexico Earthquake, Mexico, September 1986, (148-162).

Seed, H. B., Romo, M. P., Sun, J. I., Jaime, A. and Lysmer, J. (1988), "The Mexico earthquake of september 19, 1985 - relationship between soil conditions and earthquake ground motions", Earthquake Spectra, EERI, 4(4):687-729.

Discussion on paper titled: "Performance of Landfills Under Seismic Loading", by D.G. Anderson and E. Kavazanjian, Jr.
Paper No. SOA14

By: Cetin Soydemir, Vice President, Haley & Aldrich, Inc., Cambridge, Massachusetts, U.S.A.

The authors present an excellent state-of-the-practice report on the performance of solid waste landfills subjected to earthquake induced ground motions, and the current regulatory driven seismic design methodologies in the United States. The report is a timely update of the state-of-the-practice paper given by Seed and Bonaparte at the 1992 Berkeley, California Conference on the Stability and Performance of Slopes and Embankments, which set forth some key criteria currently used in seismic design of solid waste landfills.

Since geosynthetic products have become an integral element in the design of modern landfill structures, and since these elements often control the stability of modern landfills, authors' special experience with geosynthetics makes this report a more comprehensive document. The authors put on the table most of the controversial issues currently debated in the design practice, and review them objectively. In conclusion, the authors have some "good" news and some important qualifying reservations. The "good" news is that the performance record of solid waste landfills during the 1989 M7.1 Loma Prieta (i.e., San Francisco Bay Area) and 1994 M6.7 Northridge (i.e., Greater Los Angeles area) earthquakes has been generally satisfactory, even for some of those relatively close to the epicenter and having slopes as steep as 1.3H:1V, and heights up to 90 m. On the other hand, up through the present only three modern landfills with geosynthetic liner systems have been subjected to earthquake induced strong ground motions, and one of them, Chiquita Canyon Landfill experienced "significant damage", and the other two "moderate damage" during the 1994 Northridge earthquake. The "significant damage" consisted of two tears in the geomembrane liner, one approximately 3 m and the other about 23 m in length. It may be realized that to survey damage in a bottom liner would be inconclusive, where it is covered with refuse.

The intent of this review is to go over some of the pertinent points presented in the report and generate some further discussion between the authors and the body of practitioners who must deal with these issues.

o The authors argue by presenting strong motion records obtained at the Operating Industries, Inc. landfill (Greater Los Angeles area) during the 1994 Northridge earthquake that the solid waste landfills unconditionally attenuate earthquake ground motions is not a valid proposition. They also illustrate the intimate cause and effect relation between the predominant period of the base motion and the potential amplification of the motion through the waste fill. The predominant period, on the other hand, is primarily affected by the magnitude and epicentral distance of the earthquake.

o Federal EPA regulation Subtitle D requires that seismic performance be evaluated in "seismic impact zones" for new landfills and lateral expansion of existing landfills. On this matter, the reviewer would note that Massachusetts (State) Department of Environmental Protection regulations require that vertical expansion of existing landfills be also subjected to the same scrutiny. This requirement, if adopted in more seismically active regions, may have significant implications relative to the seismic response of the landfill and in turn the seismic stability issues for the proposed vertical expansion.

o The authors present a road map to conduct a simplified, approximate seismic stability analysis for solid waste landfills. The first step is the determination of the "design earthquake". In accordance with Subtitle D the U.S.G.S. map MF-2120 (i.e., the Algermissen map) can be used to establish the peak ground acceleration (PGA) in lithified earth. However, the map does not provide any guidance in determining the magnitude of the "design earthquake", which is also necessary for analysis. The reviewer would suggest that similar to PGA developing a probabilistic magnitude map (i.e., 250 year exposure period) would have merit to compliment the simplified, approximate approach.

o Based on reported laboratory test data and case histories under static loading conditions, it is well established that liners and covers typically represent the most critical elements of a landfill structure to potential instability. This is because the compacted clay and/or geosynthetic elements incorporated in the liners/covers create interfaces along which mobilization of shear resistance can be critically low. With the introduction of geosynthetic clay liners (GCL), this issue has become even more complex, where the strength parameters including "peak" and "residual" interface and midplane friction angles and cohesion intercepts are to be well understood in each case. In seismic stability analyses since both pseudo-static factor of safety and permanent deformations (i.e., yield acceleration) are directly affected by the strength parameters, a major challenge for the geosynthetic industry is the design and manufacturing of products with improved interface properties.

o Subtitle D requires that all containment structures, including liners, leachate collection systems, and covers are designed to resist the specified maximum horizontal acceleration in lithified earth. The reviewer would suggest that design of these critical components be based on a limiting (permissible) strain. Limiting strains with respect to a particular geomembrane, a compacted clay liner or a geosynthetic clay liner could be specified to maintain their respective functions. In parallel, Newmark sliding block analysis would be conducted to estimate permanent displacements under the "design earthquake". Conversion of estimated displacements to respective strains for the particular mode of instability analyzed is currently the missing link and the challenge in design analysis. Finally, the calculated strain would be compared with the limiting (permissible) strain.

Discussion on paper titled "Performance of Landfills Under Seismic Loading", by D.G. Anderson and E. Kavazanjian, Jr., Paper No. SOA14

Closure by: Donald G. Anderson and Edward Kavazanjian, Jr,

The authors thank the writer for his insightful discussion. The authors concur with the writer that additional guidance on the earthquake magnitude, or more properly on the distribution of magnitudes, associated with the peak ground acceleration portrayed on USGS map sheet MF-2120 (the "Algermissen" map) would have great merit. This information would have great value not only for seismic design of landfills, but also for liquefaction potential assessments and other geotechnical analyses which require earthquake magnitude as an input parameter.

The lack of information on earthquake magnitude is only one of the problems that must be overcome in applying the seismic hazard maps based solely on peak ground acceleration to geotechnical analyses. In selecting time histories for seismic response analyses, seismic hazard maps providing additional information on the frequency content of the earthquake motions would also have merit. Indeed, it appears that the next generation of seismic hazard maps will provide information on both peak and spectral accelerations. However, to the knowledge of the authors, there are no plans to include information on magnitude with this next generation of maps.

The authors also concur with the writer's conclusions on the need for improvement of the interface properties of geosynthetic materials. Interface properties are particularly important with respect to landfill cover systems, where low interface shear strengths combined with the potential for amplification of seismic motions can make attainment of unconditional stability difficult, even in areas which do not fall within USEPA seismic impact zones. Innovative approaches are required to develop either improved materials or improved interface geometry using currently available materials to develop increased shear strengths along geosynthetic interfaces. However, using current technology, the authors believe that permanent seismic deformation of the cover soil overlying a geomembrane interface may be treated as a maintenance problem provided there is a responsible party in charge of post-earthquake maintenance.

The writers suggestion that design of geosynthetic elements in a containment system be based upon a limiting strain is a fundamentally sound approach to maintaining the integrity of these elements of the containment system. However, it must be kept in mind that failure strains reported in the literature are often from uniaxial tension tests and are unconservative values for use as a design criteria for liner and cover system elements. Bi-axial stress states, stress cracking, and stress (or strain) concentrations can reduce allowable strains in the field to levels well below values reported in manufacturer's literature. Furthermore, it is not clear that the strain in a geosynthetic liner or cover system element can be quantitatively related to the displacement calculated in a Newmark "sliding block" analysis. The displacement calculated in a Newmark analysis is for rigid body translation of the waste or cover soil above the critical interface. Across the interface itself, there is a discontinuity in the displacement field between the overlying waste or soil mass and the underlying geosynthetic element. For a perfectly smooth interface, the underlying geosynthetic element may be able to withstand indefinitely large translation of the overlying mass without accumulating significant tensile strain.

Under field conditions, the amount of displacement in the overlying waste or soil mass that a cover or liner element can withstand without rupture will depend on such design details as penetrations, corners, and anchor points. Discontinuities and defects in the geosynthetic element may be of particular importance. Available information (EMCON, 1994) indicates that one of the liner tears at the Chiquita Canyon landfill in the Northridge earthquake initiated adjacent to an anchor trench at an extrusion welded patch over the "cut out" from a destructive seam sample recovered during construction quality assurance activities. Stress concentrations due the discontinuity in the geomembrane may have been responsible for initiation of the tear, while the stress field associated with the anchor trench may have controlled the propagation of the tear. This observation illustrates the importance of integrating design and construction quality assurance of geosynthetic containment systems.

EMCON (1994) "Northridge Earthquake Seismic Evaluation, Chiquita Canyon Landfill", report prepared for Laidlaw Waste Systems, EMCON Associates, San Jose, California, April.

CLOSURE to the discussions on the SOA paper "Deformation Characteristics of Soils and Soft Rocks Under Monotonic and Cyclic Loads and Their Relations" by Vicente Cuellar, Piere-Yves Hicher and Vincent P. Drnevich and Alaa K. Ashmawy Paper No. SOA1

By: Fumio Tatsuoka, Institute of Industrial Science, University of Tokyo, Diego Lo Presti, Politecnico di Torino, and Yukihiro Kohata, Railway Technical Research Institute (formerly IIS, Univ. of Tokyo)

The authors of the SOA paper would like to thank you very much for these stimulating and useful discussions.

<u>Closure to the discussion by Mr. Cuellar:</u>

1) In Fig. 5(c) of the SOA paper, the damping **h** of a cyclically prestrained specimen of dense Toyoura sand which was obtained from locally measured axial strains exhibits a nearly constant value for a strain range from about 0.004 % to about 0.02 %. The discussor claims that this behaviour is not acceptable theoretically. It should be noted, however, that this test result is for a <u>highly cyclically prestrained</u> specimen. It can be seen from Fig. 53 of the SOA paper that the damping **h** decreases considerably after the application of a large amount of cyclic prestraining at a certain strain level, while the amount of this change in **h** is not constant at different strain levels. Accordingly, the shape of the **h**~strain curve also changes considerable by cyclic prestraining. The authors consider that there is no sound theory which can explain quantitatively this manner of change. It seems, therefore, too early to conclude that this **h**~$d(\varepsilon_a)_{SA}$ relation shown in Fig. 5(c) of the SOA paper is "difficult to justify theoretically."

2) In Fig. 5(d) of the SOA paper, the values of damping **h** obtained from externally measured axial strains are smaller than those obtained from locally measured axial strains for some data points. The discussor considers that this result is meaningless. The authors admits that this is due partly to some inevitable scatter of data. It should be noted, however, that the values of **h** obtained from externally measured axial strains have been plotted against each respective <u>externally measured axial strain</u>, while those obtained from locally measured axial strains have been plotted against each respective <u>locally measured axial strain</u>. As stated in the text (n.b., Lines 6 to 3 from the bottom in the right-hand column on Page 856), it is possible that even when a value of **h** from externally measured axial strains is larger than the corresponding value from locally measured axial strains (as is the case), a data point (**h** and $d(\varepsilon_a)_{SA}$) based on externally measured axial strains is located below that based on locally measured axial strains, as illustrated in **Fig.** 1 of this closure.

The authors would like to apologize for having mis-labeled the h~$d(\varepsilon_a)_{SA}$) relations shown in Fig. 5a of the SOA paper. The corrected figure is shown herein (**Fig. 2** of this closure).

3) The authors agree with the discussor on that the liquefaction potential of saturated sand should be linked to the contractancy characteristics rather than to the tangent stiffness at the shear stress at a half of the peak shear stress. However, further studies will be required to reach any conclusion in this respect.

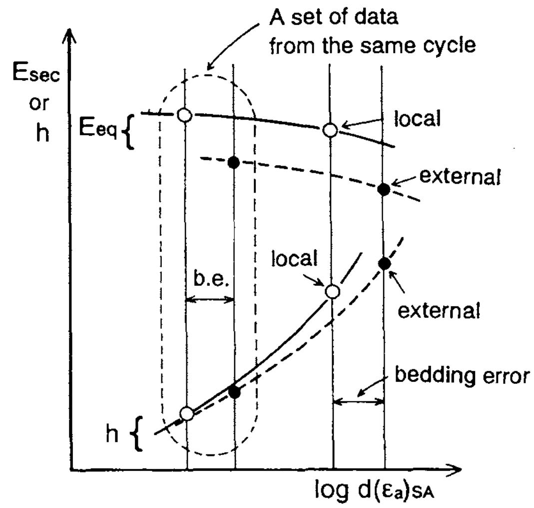

Fig. 1 Schematic diagram showing the relative positions of E_{eq}~$d(\varepsilon_a)_{SA}$ (single amplitude axial strain) curves and h ~$d(\varepsilon_a)_{SA}$ curves obtained from local and external axial strain measurements

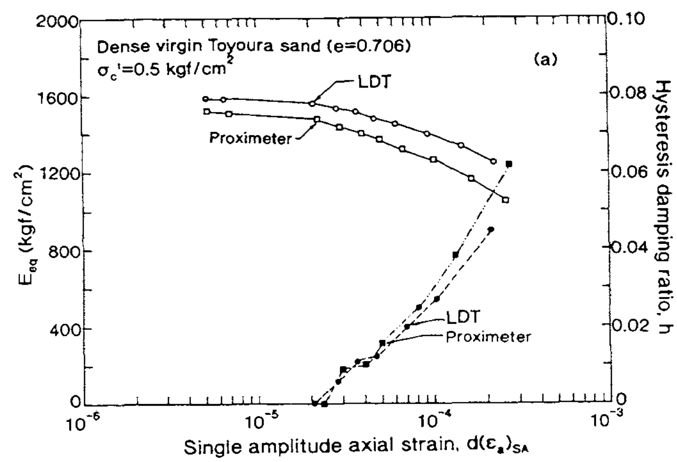

Fig. 2 E_{eq}~$d(\varepsilon_a)_{SA}$ and h~$d(\varepsilon_a)_{SA}$ relations from cyclic triaxial tests on a virgin dense specimen of air-dried Toyoura sand (the corrected figure of Fig. 5a in the original SOA paper)

4) The problem of a possible large bedding error in measured axial strains in triaxial tests on geomaterials may be very irritating to many engineers and researchers. It would be helpful, therefore, if a guidance is available which would suggest whether a local axial strain measurement is really imperative for a given test. **Fig. 3a** of this closure shows the values of the initial Young's moduli E_{max} defined for a range of axial strain smaller than about 0.001 % of four types of cohesive soils, which were obtained from a series of CU triaxial compression tests (TC tests) and CD TC tests (only Kanto loam) performed at IIS, University of Tokyo. Undisturbed samples were reconsolidated to each respective estimated field stress state. Both values of E_{max} obtained from axial strains measured externally and locally by means of the local deformation transducer (LDT) are shown. The values of E_{max} have been plotted against the axial strain $(\varepsilon_a)_o$ observed during anisotropic consolidation or isotropic (only for Kanto loam) starting from a confining pressure equal to about 0.3 kgf/cm². For the Kanto loam

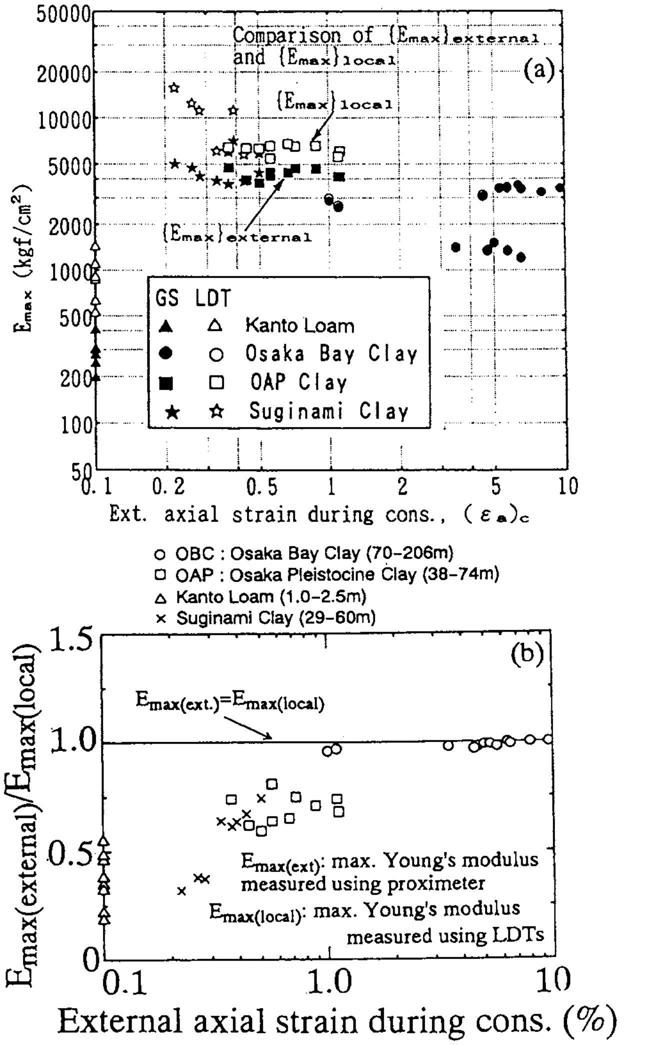

Fig. 3 a) Relationship between initial Young's modulus E_{max} from local and external axial strain measurements and axial strain $(\varepsilon_a)_c$ during consolidation of cohesive soils, and b) relationship between the ratio of the E_{max} values from local and external axial strain measurements and $(\varepsilon_a)_c$

specimens, the values of $(\varepsilon_a)_c$ were very small, less than 0.1 %, thus the E_{max} values were plotted arbitrarily at $(\varepsilon_a)_c$= 0.1 %. It may be seen that for many data points, the E_{max} values obtained from locally measured axial strains are larger than each respective value obtained from externally measured axial strains. It may be seen, however, that the difference between each pair of E_{max} values is not uniquely linked to the E_{max} value itself. That is, the differences are not a function of the stiffness. **Fig. 3b** of this closure shows the ratios of the E_{max} values obtained from locally and externally measured axial strains plotted against $(\varepsilon_a)_c$. It can be seen that the ratio is a rather unique function of $(\varepsilon_a)_c$; that is, the ratio increases consistently from a value of less than unity to around unity as $(\varepsilon_a)_c$ increases to about 1 % or more. It is very important that the following set of data which is similar to that test result (Fig. 3b of this closure) obtained at IIS, Univ. of Tokyo has been obtained independently at Politechnico di Torino. That is, the E_{max} values obtained from axial strains measured externally and locally with LDTs in CD TC tests on Pisa clay undisturbed samples isotropically or anisotropically consolidated to respective in-situ vertical stress and axial strains during consolidations $(\varepsilon_a)_c$ are shown below;

E_{max} (MPa) Local ε_a	External ε_a	Ratio	$(\varepsilon_a)_c$ (%)
60	50	0.83	0.63
58	41	0.71	0.72
60	56	0.93	1.86

The test results shown above suggest that for the case where it is expected that $(\varepsilon_a)_c$ is less than about 1%, an appropriate type of local gauge should be used to obtain meaningful stiffness values.

Closure to the discussion by Mr. Hicher:

1) The authors appreciate that the importance of local strain measurement for triaxial tests on stiff geomaterials has already been pointed out by Hicher and his colleagues in 1981 (El-Hosri et al., 1981), as referred by Tatsuoka and Kohata (1995).

2) The discussor pointed out that for cohesive soils, or relatively soft geomaterials, stiffness even at very small strains of, say less than 0.001 %, is noticeably strain rate-dependent, as demonstrated in Fig. 2 of the discussion. The authors have also noticed this point as discussed by Tatsuoka and Shibuya (1992) and Tatsuoka and Kohata (1995). On the other hand, it is very difficult to imagine that the stiffness at small strains continues to increase as the strain rate increases to a very large value. The authors feel that for most types of geomaterials, the rate of the increase in the small strain stiffness for a given rate of the change in the strain rate starts decreasing when the strain rate exceeds a certain value. The data presented in Fig. 2 of this discussion also show a tendency of the decreasing rate of the increase in the small strain stiffness with the increase in the strain rate.

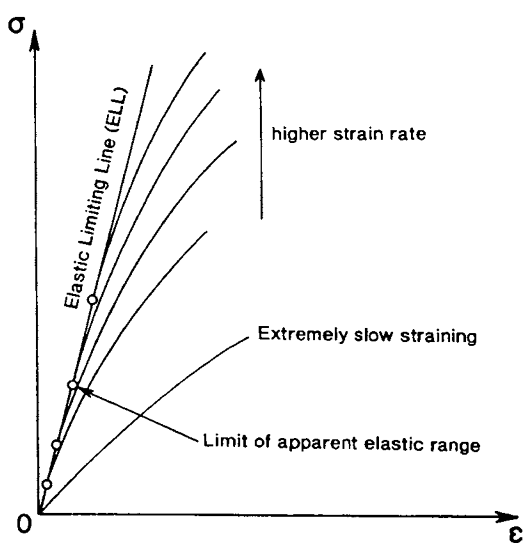

Fig. 4 Schematic diagram explaining the elastic limiting line (reproduction of Fig. 5.29 of Tatsuoka and Shibuya, 1992).

Tatsuoka and Shibuya (1992) propose the concept of "Elastic limiting line (**Fig. 4** of this closure) to explain these features described above. When following this concept, the elastic range disappears when the strain rate becomes lower than a certain critical value, while the elastic limit strain appears only when the strain rate exceeds this critical strain rate. The elastic limit strain increases as the strain rate increases. When the strain range for which the secant stiffness is defined is larger than the elastic limit strain, then the secant stiffness depends on the strain rate. Therefore, in general, the effect of strain rate on the stiffness becomes more noticeable as the strain rate decreases and vice versa. This point is seen from the test result of Pisa clay shown in Fig. 45 of the SOA paper. Also from the data presented by the discussor (Fig. 2 of the discussion), the ratio of E_{sec}(f= 0.2 Hz)/E_{sec}(f= 0.002 Hz) is about 1.24 at ε_a= 0.001 %, while it increases to 1.7 at ε_a= 0.01 %. A possible strain rate effect on the stiffness even at small strains (ε_a= 0.001 %) is compatible with the ELL concept if one considers that the elastic threshold strain ε^e_t for certain soils (i.e., soft clays) is very low, for example, ε^e_t= 0.0001 % to 0.001 %.

The authors admit that further research will be needed to validate (or invalidate) this concept.

3) The discussor claims that the partially drained tests as reported in the SOA paper are not element tests, since the stress-strain conditions within a specimen are not homogeneous. The authors fully agree on this point. That is, damping values measured in a nominally drained, but actually partially drained, cyclic triaxial test are not material properties, but they are the response of a system under certain boundary conditions. The authors attempted to understand why the damping **h** of a specimen of sedimentary soft rock and clay under partially drained conditions can become much higher than those measured under undrained conditions. It should be noted that damping (or dissipation of energy) resulting from the interaction (or relative movement) between pore material and soil skeleton becomes very important in many cases; for example, high-frequency laboratory tests such as resonant-column tests on water-saturated soil or seismic behaviour of a water-saturated rockfill dam. This type of damping is not a material property of geomaterial skeleton (or geomaterial itself), but it is certainly a intrinsic property of a geomaterial skeleton-pore water system. The authors consider, therefore, that the understanding of the damping of water-saturated geomaterial under partial drained conditions is of not only academic interest but also practical importance.

4) The authors should apologize that Fig. 47 of the SOA paper is not consistent to the plotting method of the results of cyclic triaxial tests in this SAO report, in which damping values have been plotted against strain. Perhaps, Fig. 47 of the SOA paper should be modified as **Fig. 5** of this closure. It should be noted, however, that even when compared at the same strain amplitude, the damping increases by creep effects.

5) The authors fully agree with the discussor on the importance of considering possible large effects of sample disturbance. The authors feel that the degree of sample disturbance is not a simple function of unconfined strength (or unconfined compressive strength) of a given geomaterial, which may represent the resistance against sample disturbance during coring and handling. It seems that it also depends on the sampling method (or the disturbing forces during sampling). This infer is based on the fact that although the problem of sample disturbance becomes less serious in tube sampling of cohesive soil compared when compared with that in tube sampling of cohesionless soil, it becomes serious again in rotary core tube sampling of sedimentary soft rock. It is attempted in **Fig. 6** of this closure to describe this feature very conceptually. It seems that this rather complicated situation is one of the reasons for the lack of a consensus on the sample disturbance problem. Systematic studies will be required to qualitatively understand this problem.

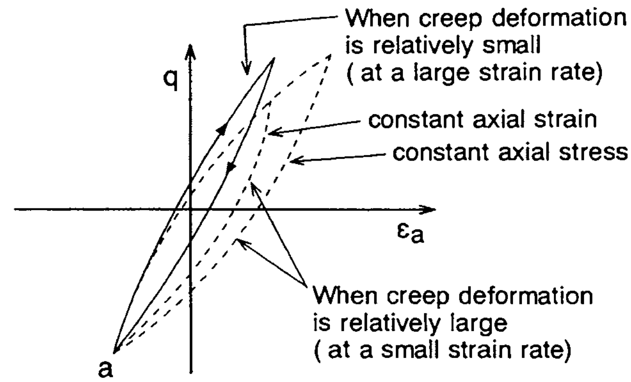

Fig. 5 Schematic diagram to explain the creep effect on damping (modified from Fig. 47 of the SOA paper)

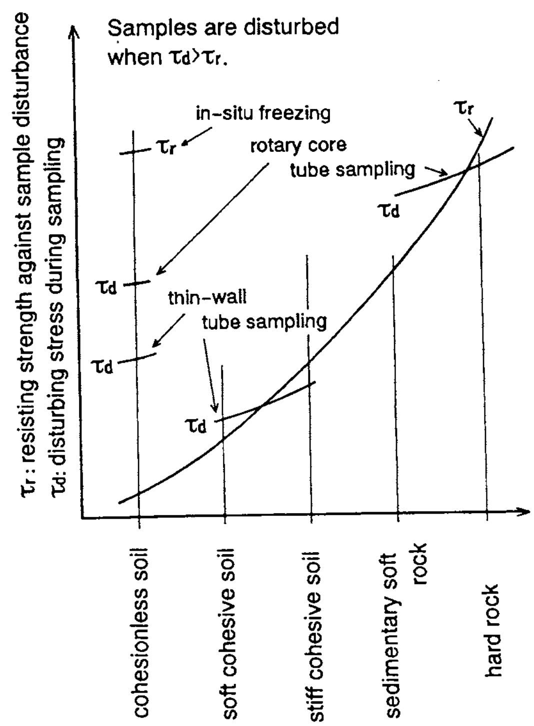

Fig. 6 Conceptual diagram to explain the effect of sample disturbance for a wide variety of geomaterials

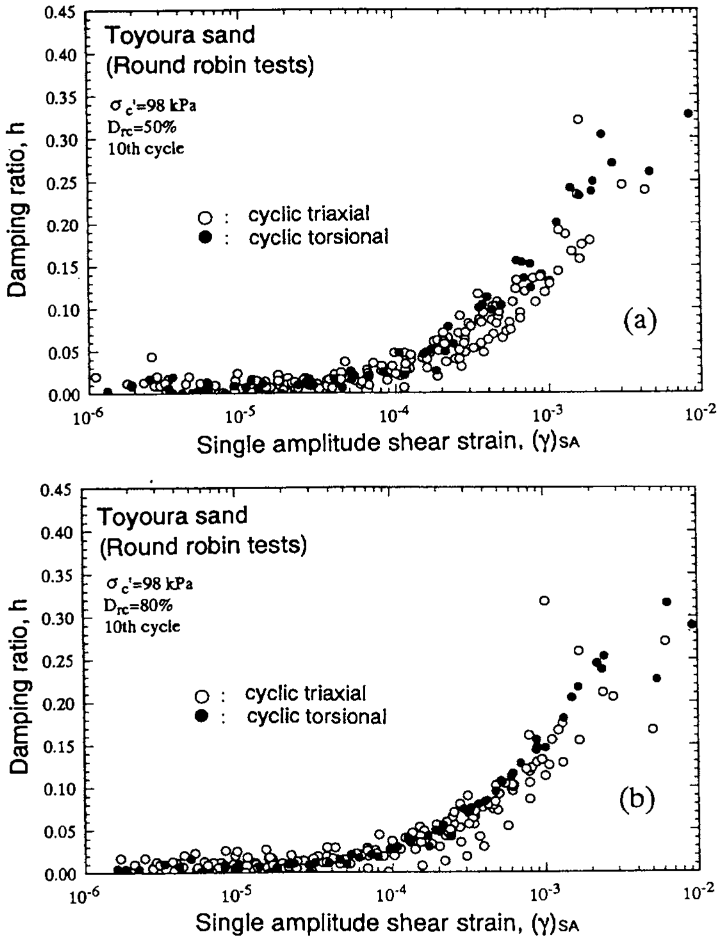

Fig. 7 Comparison of damping of Toyoura sand between triaxial tests and torsional shear tests from the round robin test program (summarized by Dr. S. Yamashita, Kitami Institute of Technology)

Closure to the discussion by Prof. Drnevich and Mr. Ashmawy:

1) The discussors suggest one more factor which may affect the damping of geomaterials, which is "loading stress path." As pointed out by the discussors, Hardin (1965) showed that the damping at small strains of dry Ottawa sand in axial loading is approximately 3/4 that in torsional loading. It should be noted, however, that that set of data had rather large scatter, while no theoretical reason was given. **Fig. 7** of this closure shows the damping values **h** of Toyoura sand obtained from the round robin cyclic triaxial tests, which are presented in Fig. 11 of the SOA paper, and those from cyclic torsional shear tests on a hollow cylindrical specimen, which is the other series of the round robin tests. The specimens were isotropically consolidated for both types of tests. Scatter of the data prevents a rigorous comparison of **h** between the two types of tests (n.b., the effect of bedding errors on some **h** values from some triaxial tests is another factor which makes this comparison complicated). However, such a tendency as that at strains less than 0.01 %, the **h** values from triaxial tests are systematically lower than those from torsional shear tests is not noted. More rigorous and delicate tests will be required to reach any conclusion in this respect.

2) The authors fully agree with the discussors on that the loading frequency does not directly control the damping **h** of geomaterials. The tests on sedimentary soft rock, the results from which are presented in Figs. 41 and 46 of the SOA paper, were stress-controlled ones using a sinusoidal wave form of cyclic stresses, while those on stiff clay, the results of which are presented in Figs. 44 and 49 of the SOA paper, were strain-controlled ones using a triangular wave form of cyclic strains. These two types of tests are not consistent to each other, since the strain rate is not constant in the former tests, while it is constant in the latter tests. The authors admit that this loading conditions should be consistent in studies which delicately investigates damping at small strains of geomaterials.

As argued by the discussors, in both static and dynamic cyclic tests performed in the current practice, the loading frequency is kept constant when changing the strain amplitude. In such tests, strain rate increases with strain level.

3) The discussors argue that a certain overlap exists between the strain rates which are typically used in quasi-static cyclic and dynamic tests. However, for a similar wave form, the strain rates are generally larger in the dynamic tests by the order of one to two than in the equivalent static tests.

Reply to Discussions of "Recent Advances in Centrifuge Modeling of Seismic Shaking" (SOA8)

B.L. Kutter
Department of Civil and Environmental Engineering
University of California
Davis, California

I would like to thank Dr. Law and Dr. Madabhushi for their interest in my state-of-the-art paper. They point out several important issues that were not covered completely. The contributions of Madabhushi and Law on the effect of fluid viscosity on mechanical properties, the VELACS project, and control algorithms for multiple actuator shakers form important supplements to my paper. I would like to expand on their discussion, however, on the topics of desirable base motions, boundary effects, and data acquisition.

IMPORTANCE OF REALISTIC GROUND MOTIONS

Madabhushi questions the need for using "realistic" earthquake ground motions in centrifuge shakers. He argues that using a number of single frequency wave trains spanning the frequency range of interest is sufficient. It is true that this type of test will provide academically interesting data. If the shaking event triggers nonlinear or permanent deformations, however, one series of single frequency wave trains may densify the soil and alter the response in subsequent wave trains. Field data is available or can be obtained to study the linear seismic response of full scale structures during relatively frequent small earthquakes. Centrifuge testing provides a unique contribution of data to investigate the non-linear response to large earthquakes; superposition does not apply to non-linear phenomena.

Law, does not disagree that use of realistic earthquake motions are important, but he makes the point that the use of sinusoidal input motions should not be discouraged. Much can be learned, especially for linear problems, by using pure sinusoidal motions.

Using the new servo-hydraulic actuation system on the large centrifuge at Davis, we have imposed small amplitude motions of varying frequency content; often continuously varying frequency sine sweeps to study the linear response of a model. This has been found to be much more efficient than conducting many tests using a single frequency. "Step waves" have also been found to be quite useful for studying approximately linear behavior. Following the characterization of the small strain behavior of the models, a large amplitude, "realistic earthquake" is usually imposed. Facsimiles of ground motions from Kobe and Loma Prieta Earthquakes have been imposed on 2 ton submerged sand samples. To date, base accelerations as large as 25 g (model acceleration) have been achieved. Servo-hydraulic actuation systems have the flexibility to impose sine waves, sine sweeps, step waves, as well as more realistic motions.

Non-linear effects such as pore water pressure build up and liquefaction might not be correctly modeled by uniform stress cycles. There are empirical rules for converting realistic earthquake motions to an "equivalent" number of uniform cycles; but these rules ought to be the subject of research, not the basis for a research approach. Calibration of constitutive models may be performed using uniform cycles, but constitutive models also have a variety of rules which determine the hysteretic behavior during stress reversals. These rules might often result in dramatically different predictions for uniform motions and realistic motions. Verification of constitutive model predictions for uniform cycles does not verify predictions for realistic motions.

Law cites an example in my paper where I show apparent differences in character of ground response due to realistic motions and sinusoidal motions. He correctly argues that I have not proven that realistic earthquakes produce different results than sinusoidal ones. On the other hand, the burden of proof ought to be placed on researchers that promote the use unrealistic earthquake motions.

Madabhushi explains that a new actuation system (SAM) is being developed at Cambridge. It is not clear that their new machine will be capable of providing single frequency sine waves, variable frequency sine sweeps, or both. If SAM is capable of producing variable frequency sweeps, it will be a very useful device. One test with continuously varying frequency can provide more information than several tests using a single frequency. Real earthquake motions, however, contain amplitude modulation as well as frequency modulation. A swept sine wave provides data to investigate the effect of different frequencies, but little information about the effect of varying amplitude.

The spectrum of possible ground motions is very large; it is arguably desirable to limit the scope of investigation to realistic motions.

BOUNDARY EFFECTS AND MODEL CONTAINERS

Madabhushi mentions the importance of continued study of the shear sheets at the end of the container. At Davis, we have relied on the work at Cambridge, and have implemented the shear sheet concept into three different types of model containers.

As described in the paper, for the small centrifuge at Davis, we have developed a fairly complicated "hinged plate" container (HPC) which uses much fewer rings than the laminar boxes developed at Caltech, Colorado, and RPI. The HPC, however, allows rotation of the ends of the rings to provide reasonable simulation of vertically propagating shear waves using fewer rings than are needed for a laminar box. Law correctly points out that the HPC is inferior to the laminar boxes in terms of the continuity of displacement along the container sides. Intuitively, it seems that elimination of the bearing stress discontinuity on the end boundary is a good trade for increased shear stress discontinuity on the side boundaries. The HPC supports each ring independently; the roller bearings are not all clamped together with a preload as for laminar boxes. It is believed that this system provides reduced frictional restraint.

For the large centrifuge shaker, an FSB container with shear sheets on the end is being used. We use 20 durometer, 12 mm thick neoprene rubber between each of six rings made from hollow rectangular aluminum tubing. Each rubber layer can easily deform 12 mm in shear. While this is not unlimited, it does allow an average cyclic or permanent shear strain of about 10% with minimal, measurable, approximately elastic resistance.

INTERACTION BETWEEN MODEL AND SHAKER

In the paper, it was recommended that the interaction of the soil, container, actuators and reaction mass ought to be evaluated. Law argues that if the "rigid" sample base plate motion is measured, and used as input for analysis of the model behavior, the reaction mass motion can be ignored. His argument neglects energy that may be transmitted from the model back through the base plate and absorbed by the reaction mass. In effect, the base plate is a "transmitting boundary". When a displacement boundary condition is specified at the base of a numerical model, the transmission of energy across the base is neglected. The importance of this issue needs to be investigated.

DATA ACQUISITION

Dr. Law's comments regarding data acquisition are also very important. In this regard, the large centrifuge at UC Davis uses an onboard computer to control the new shaker, to digitize experimental data and store the data on a hard disk. The onboard computer contains a commercial 32 channel analog to digital converter and 32 channels of anti-aliasing filter amplifiers, with cutoff frequencies and gains that can be adjusted by software without stopping the centrifuge. Data can be obtained at frequencies of 320 kHz or less. The human interface to the onboard computer is made via an offboard computer in the control room, remote control software, and an ethernet network.

Reply to Discussion by Mr. Fabrizio Felli on State-of-the-Art #9, "Simple Physical Models for Foundation Dynamics"

By John P. Wolf

The author would like to thank discusser for his interest in the paper. The following two observations are made:

1) It is true that significant deviations for the dimensionless spring coefficient $k(a_0)$ for large a_0 occur (a_0>6 in Figs. 3 and 10c). However, these differences are less serious than they seem. Note that the dimensionless damping coefficient $c(a_0)$ is multiplied by a_0. For large a_0 the deviations in $k(a_0)$ are negligeable since the product a_0 $c(a_0)$ dominates. This is further substantiated by viewing the magnitude and phase angle of the dynamic-stiffness coefficient where the deviations become small. A thorough discussion on this aspect is contained in Ref. 25 of the paper.

2) Extensions of simple physical models to other cases are very welcome which have to be evaluated systematically before they can be used.

Discussion on paper titled: "Simple Physical Models for Foundation Dynamics", by John P. Wolf, Paper No. SOA9

By: Marshall Lew, Law/Crandall, Inc., Los Angeles, California, USA.

During the state-of-the-art presentation, the author made the comment that the simple model techniques described in the paper could be used for nonlinear behavior. The author later qualified this statement saying that the nonlinearity would be limited to the interface between the structure and the soil, or only in the structure itself. For any significant earthquake ground motions, it should be expected that the soils will begin to exhibit nonlinear behavior sooner than the structure will, except perhaps in the case where the structure is founded on extremely hard rock.

Reply to discussion by Mr. Marshall Lew on paper "Simple Physical Models for Foundation Dynamics"

The author thanks the discusser for his interest in the paper.

For earthquake excitation a non-linear free-field analysis will provide equivalent elastic moduli which can then be used for the actual interaction analysis using simple physical models. The latter assumes linear behaviour of the soil, i.e.. additional non-linear effects in the soil caused by the interaction with structure are disregarded.